NATIONAL MAPPING COUNCIL OF AUSTRALIA

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Third Edition

STANDARD SPECIFICATIONS AND RECOMMENDED PRACTICES FOR HORIZONTAL AND VERTICAL CONTROL SURVEYS

Third Edition

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STANDARD SPECIFICATIONS AND RECOMMENDED PRACTICES FOR HORIZONTAL AND VERTICAL CONTROL SURVEYS

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STANDARD SPECIFICATIONS
HORIZONTAL AND VERTICAL CONTROL

1. INTRODUCTION

1.1 These standards apply to control surveys for determining the geographical and rectangular coordinates and heights of marked stations throughout Australia and its Territories.

2. CLASSIFICATION

2.1 Control surveys are classified in accordance with the design pattern laid down and by the degree of accuracy planned and achieved in accordance with these specifications. The authority undertaking a control survey shall make appropriate and adequate check measurements for the purpose of confirming the classification of the survey.

3. HORIZONTAL CONTROL

3.1 Horizontal control will normally be provided by Doppler satellite observations, triangulation, traverse, trilateration or a combination of these methods.

3.2 Datum for Coordinates

For the Australian mainland and Tasmania, all geodetic coordinates shall be computed on the Australian Geodetic Datum. For islands and the Australian External Territories that cannot be connected to the Australian Geodetic Datum by direct measurement, World Geodetic System 1972 is to be used - see Schedule 2.

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Special High Precision surveys should normally be of limited extent and be specifically undertaken for purposes where the utmost precision is required.

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First order surveys are carried out to the specified accuracy as the highest accuracy that is economically reasonable and shall normally be executed along lines of primary control as approved from time to time by the National Mapping Council. First order surveys may also be used in conjunction with second order surveys to provide a fundamental geodetic network over any particular area.

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Second order surveys shall normally be used to subdivide the area between first order control for the purpose of providing a fundamental geodetic network of first and second order stations over the entire area to be controlled.

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Third order surveys shall normally be used to subdivide the area between first and second order stations for the purpose of controlling a network of third and fourth order stations over the entire area, at distances apart of approximately 10 to 15 kilometres.

3.3.5 Fourth Order

Fourth order surveys shall be used to provide control for mapping where the third order control is too far apart.

3.4 Classification - Doppler position fixing

The absolute coordinates determined by Doppler point positioning are classified thus:

Class	Reference System	Characteristics
1.	Precise Ephemeris	. minimum of 35 acceptable precise ephemeris passes . expected accuracy is 1.5 metres in each component at 90% confidence interval
2.	Precise Ephemeris	. between 20 and 34 acceptable precise ephemeris passes . expected accuracy is 2.0 metres in each component at 90% confidence interval
3.	Precise Ephemeris	. between 12 and 19 acceptable precise ephemeris passes . expected accuracy is 2.5 metres in each component at 90% confidence interval.
4.	Precise Ephemeris	. between 6 and 11 acceptable precise ephemeris passes. . expected accuracy is better than 4 metres in each component at 90% confidence interval.
5.	Onboard Ephemeris	. minimum of 20 acceptable onboard ephemeris passes (75 passes optimum) . expected accuracy is about 10.0 metres in each component at 90% confidence interval.

6.	Onboard Ephemeris	. less than 20 acceptable onboard ephemeris passes. . solution is unreliable.

where the two-sided 90% confidence interval is equivalent to about 1.65 standard deviations.

3.5 Accuracy - Requirements

3.5.1 Horizontal

Horizontal control surveys shall conform to the following standards of accuracy:

Order of Survey	Standard errors in adjusted values of lines joining adjacent stations not to exceed:	
	Length (Parts per million)	Azimuths (seconds)
Special high precision	Substantially more accurate than required for first order	
First	7.5	1.5
Second	15	3
Third	30	6
Fourth	0.25 - 0.50 mm at scale of mapping	

and fundamental geodetic networks shall conform to the standards of accuracy approved by the International Association of Geodesy as set out in Schedule 1.

3.5.2 Heights

The heights of horizontal control stations shall be determined to such a degree of accuracy as is necessary for the purpose for which the station is being established.

4. VERTICAL CONTROL

4.1 Vertical control will be provided by differential levelling, or in special situations such as at river crossings by trigonometrical heighting techniques, to first, second, third or fourth order accuracy using appropriate survey equipment and practices etc.

4.2 Datum for Heights

Heights should be based on the Australian Height Datum. When connection to the Australian Height Datum is not possible, heights shall approximate as closely as possible to heights above mean sea level, and the datum used should be carefully defined.

4.3 Accuracy - Requirements

4.3.1 Vertical

Vertical control surveys conform to the following standards of accuracy:

Class of Survey	Accuracy Requirement
1st Order	The two levellings of each section between permanent bench marks shall not differ by more than $4\sqrt{K}$ mm where K is the distance in kilometres between bench marks measured along the levelling route. Circuit closures shall not exceed this same limit, where K is the length of the circuit in kilometres along the levelling route.
2nd Order	The two levellings of each section between permanent bench marks shall not differ by more than $8.4\sqrt{K}$ mm where K is the distance in kilometres between bench marks measured along the levelling route. Circuit closures shall not exceed this same limit, where K is the length of the circuit in kilometres along the levelling route.
3rd Order	The two levellings of each section between permanent bench marks shall not differ by more than $12\sqrt{K}$ mm where K is the distance in kilometres between bench marks measured along the levelling route. Circuit closures shall not exceed this same limit, where K is the length of the circuit in kilometres along the levelling route.
4th Order	If the levelling is two-way the two levellings of each section between permanent marks shall not differ by more than $18\sqrt{K}$ mm where K is the distance in kilometres between bench marks measured along the levelling route. If the levelling is one-way metric-foot using dual faced levelling staves the two levellings of a section, one in metres and the other in feet shall not differ by more than this same limit. Circuit closures, where circuits are made up wholly of 4th order levelling, shall not exceed this same limit, where K is the length of the circuit in kilometres along the levelling route. Where levelling is one-way, either metric-foot or metric only, the extremities of the traverses will be connected to traverses of a high order and these circuit closures shall not exceed $36\sqrt{K}$ mm where K is the length of the circuit in kilometres along the levelling route.
Heights for Topographic Mapping	Sufficient to control contouring of the area to be mapped.

4.4 Accuracy - Evaluation after Completion of Field Work

Levelling may be classified by international standards by evaluation of the probable total limited error per kilometre. See International Association of Geodesy: Resolutions Relative to Precision Levelling, at Schedule 1, Part 2.

5. ATTACHED SCHEDULES

5.1 International Association of Geodesy specifications for -

Part 1 - fundamental networks in geometric geodesy;
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Schedule 1

Part 1

INTERNATIONAL ASSOCIATION OF GEODESY

SPECIFICATIONS FOR FUNDAMENTAL NETWORKS IN GEOMETRIC GEODESY

(Resolution No 6 adopted by the IAG at its XIIIth Assembly, 1963)

> "The International Association of Geodesy
>
> considering that fundamental geometric networks should be established with uniform accuracy sufficiently high for various scientific purposes,
>
> resolves that specifications for the establishment of such networks, which are given in the final report on specifications of Special Study Group No 14, be adopted as minimum standards."

EXTRACT FROM REPORT OF SPECIAL STUDY GROUP NO 14

1. GENERAL SPECIFICATIONS

1.1 Geodetic networks which form the unitary basis for national geodetic operations or are established as international links which enable scientific investigations to be made on an international basis as for instance as regards the determination of the dimensions of the earth are defined as fundamental networks.

The fundamental network should be established with such an accuracy, that in the adjusted net the standard error (s.e.) of the relative position of neighbouring stations in an absolutely oriented system never exceeds 1:100 000$\sqrt{S/30}$, S denoting the distance between the stations in km.

1.2 A fundamental network should be formed in such a way that every part of it is well checked through geometrical conditions. Thus, in triangulation and trilateration single chains should not be allowed, but the network should be built up by double chains, chains in polygons or preferably as area nets. Traverses should always be arranged in systems of polygons.

1.3 In case the scale of the network is determined by means of length measures like tapes or wires, calibrations of the measures on international metre standard has to be carried out, preferably by using a standard base line adequately standardised. In case the scale of the network is determined by electro-optic or radar-type methods the value of the velocity of light should be used as accepted by the Association.

2. RECOMMENDATIONS OF STANDARDS FOR VARIOUS MEASUREMENT METHODS

2.1 Horizontal angle measurements

In order to discover lateral refraction influence and to diminish systematic errors due to refraction the measurements of a horizontal angle should preferably be carried out under various atmospheric conditions and should be distributed on a number of different days. In triangulation work the accuracy as computed by the Ferrero formula should give a s.e. of station adjusted direction less than 0.4". Triangle closures larger than 2.5" should be rejected. In traverse work and in connection with trilateration work, where the Ferrero formula is not applicable the s.e. of station adjusted direction as computed from the station adjustments should average less than 0.35".

2.1.1 Ferrero's Formula

$$\text{s.e.} = \sqrt{\frac{\Sigma\xi^2}{3n}}$$

where ξ = triangular error

= Σ 3 angles - (180° + spherical excess)

and n = number of triangles

2.2 Base measurements with base extension nets

Base measurements should be arranged in a way to exclude as far as possible uncheckable systematic sources of errors. Tapes or wires should be calibrated before and after the base measurements according to 1.3. At least three different measurements should be used in each direction. Base nets should be given a configuration which with regard to the local circumstances as far as possible exclude systematic effects due for instance to refraction. It is recommended to plan these measurements to give a s.e. less than 1:400 000 of the triangle side (the extended base). The s.e. should never exceed 1:200 000.

2.3 Distance Measurements

The measurements should be distributed in at least two different days. The type of equipment and methods used should be capable of accuracy (s.e.) of 1:300 000 when used for measurements substituting bases and base nets, and of 1:200 000 in traverse and trilateration work. The meteorologic observations during the measurement period should be arranged in a way to secure a corresponding accuracy in the determination of the refractive index. Checking of frequency and instrument constants should be duly carried out in connection with the measurements. Special precautions, which depend on the type of instrument used, will have to be taken in order to reach the required accuracy. Measurements substituting bases and base nets should preferably be arranged in such a way that independent checks can be had for instance by measuring the sides and the angles of a triangle or some other closed figure. A trilateration network should preferably be strengthened by suitable angle measurements for instance along traverses joining the Laplace stations.

2.4 Astronomic measurements of azimuth and longitude at Laplace stations should be arranged to exclude as far as possible systematic influences. The stations should preferably be observed by pairs with simultaneous reciprocal azimuth determinations. The determinations of azimuth as well as longitude should be distributed in at least three different nights. Longitude should be determined as the difference of longitude from a longitude reference station, where corresponding measurements have to be made in such a way that systematic effects as far as possible are eliminated in the difference. The measurements should have such an accuracy, that the s.e. of the expression $A - L \sin B$ (A denoting azimuth, L longitude and B latitude) as computed from the station measurement data is less than 1".

2.5 Distribution of bases (measured sides) in triangulation nets

The density of measured sides has to be deduced in such a way that even in the weakest parts of the net the accuracy of side length corresponds to the general specification on relative position of neighbouring stations. Since this accuracy will be influenced by many factors such as accuracy of the different measurements introduced in the network and of the local figure of the net, no very simple rule for determination of this density can be expressed. It is recommended, that the side which is estimated to be the weakest one in between two bases, as these have been planned, be tested by computation, from a narrow part of the net between two bases, of the approximate weight the side length will get in the adjusted net, making it possible to estimate the s.e. to be expected. Since however in triangulation networks scale errors will get a systematic character over parts of the net, it is recommended to apply the method with a wide margin.

A simpler procedure for the estimate is however very desirable. A first estimate can be had by means of the well-known strength of figure expression completed with regard to the inaccuracy in the measured sides. The criterion might be expressed as:

$$100 > R + \frac{50}{n_a^2} + \frac{50}{n_b^2}$$

R being the strength of figure value computed between two proposed bases, the s.e. of which are estimated to 1:100 000 n_a and 1:100 000 n_b respectively.

2.6 Distribution of Laplace stations

The general accuracy specification corresponds reasonably to a largest permissible s.e. in azimuth of a triangle side of 2". Using an estimate of the accuracy, which is kept in the various measurements introduced in the network, a test of the azimuth accuracy of the weakest side between two proposed Laplace stations can be made in a similar way as described in 2.5. To avoid, however, the effect of possible systematic orientation errors over wide parts of the net, it is recommended to arrange Laplace stations considerably denser than indicated by this method. However, in traverse and triangulation nets as well as trilateration nets with angle measurements along traverses joining the Laplace stations, a simpler rule might be sufficient, giving the permissible number of azimuth transporting angles between two Laplace stations as a function of the accuracy of horizontal angle measurements and astronomic measurements. As a reasonable approach, corresponding to the accuracy rules of 2.1 and 2.4 the highest number of azimuth transporting angles between Laplace stations might be put as 8 - 10 in triangulation and 6 or less in traverse work.

2.7 Old networks, which are found too weak, may frequently be reinforced to meet the accuracy requirements given above by introducing additional distance measurements and Laplace stations. It is however necessary to make perfectly sure that the stations involved in such additional measurements are unchanged since they were used in the original triangulation. Usually this will demand a check by remeasurement of a suitable part of the net including the stations in question and by comparison with the previous results.

Schedule 1

Part 2

INTERNATIONAL ASSOCIATION OF GEODESY

RESOLUTIONS RELATIVE TO PRECISION LEVELLING

(Resolutions adopted by the IAG at its VIIIth Assembly, 1948)

1. EVALUATION OF THE PRECISION OF A METHOD OF LEVELLING

1.1 Accidental and systematic errors

Levelling is affected by two classes of errors, called accidental and systematic, independent of each other, and having the following characteristics:

Accidental errors - These are due to causes acting independently on all successive levelling observations. They obey the laws of Gauss. They are characterized by the coefficient:

η, called the probable accidental error per kilometre such that the probable accidental error for any distance L is $\eta\sqrt{L}$.

Systematic errors - These are due to causes acting in a similar manner on successive or adjacent levelling observations. They do not obey the laws of Gauss. Their effects become accidental only for distances L exceeding a certain limit Z, of the order of several tens of kilometres. They are characterized by a coefficient:

ζ, called the probable accidental limiting value (per kilometre) of the systematic error such that the probable systematic error for a distance $L \geqslant Z$ is $\zeta\sqrt{L}$.

For a distance $L < Z$, the probable systematic error in $\zeta L\sqrt{L}$, the factor ζL increasing from 0 to ζ as L increases from 0 to Z.

Total error - Accordingly the combined influence of these errors is characterized by a unique coefficient:

τ, called the probable accidental limiting value (per kilometre) of the total error or the probable total limiting error per kilometre such that the probable error for a distance $L \geqslant Z$ is $\tau\sqrt{L}$, and therefore, $\tau^2 = \eta^2 + \zeta^2$

For a distance $L < Z$, the probable total error is $\tau L\sqrt{L}$, the factor τL increasing from η to τ as L increases from 0 to Z ($\tau L^2 = \eta^2 + \zeta\ L^2$).

1.2 Notation

Let:

R = distance between two consecutive bench marks;
L = the length of one of the segments into which the net can be divided;

These segments can be made up in many different ways, by grouping or by breaking up the links, which connect two junctions of the net. A single segment can comprise some elements separated on the group but associated in the computations.

F = the perimeter of a closed circuit;

The closed circuits can be made up in many different ways, by grouping several elementary closed circuits of the net.

R_m, L_m, F_m = the mean values of the elements R, L, F;
n_R, n_L, n_F = the number of elements R, L, F;

ρ, λ = the discrepancies, noted in the distances R, L, between the results of two component levellings, if the levelling considered is the mean of two such levellings, assumed to be independent;

μ = the difference, for a continuous segment of length L, between the extreme ordinates of the mean straight line drawn (with minimum deviation) across the graph of accumulated discrepancies;

ϕ = the closure of a closed circuit of perimeter F, after applying the correction for the lack of parallelism of the terrestrial level surfaces;

γ = the correction for the segment L derived from the rational adjustments of the net;

Here the segment L must be formed only of elements belonging to the same link and taken in the same direction along the link.

$\Sigma R, \Sigma L, \Sigma F, \ldots \Sigma\rho^2, \Sigma\frac{\rho^2}{R}, \ldots$ = the sums of the elements $R, L, F, \ldots \rho^2, \frac{\rho^2}{R}, \ldots$;

mean $\frac{\rho^2}{R}$, mean $\frac{\lambda^2}{L}$, ... = the means of the elements $\frac{\rho^2}{R}, \frac{\lambda^2}{L}, \ldots$

These means can be calculated by assigning to the elements

$\frac{\rho^2}{R}, \frac{\lambda^2}{L}, \ldots$ weights respectively equal to 1, R or L ...

R, L, F are expressed in kilometres; $\rho, \lambda, \mu, \phi, \gamma$, in millimetres.

The various probable errors η, ζ, τ are expressed in millimetres per kilometre.

1.3 Formulas for the evaluation of the precision

Coefficient τ, the probable accidental limiting value (per kilometre) of the total error, or probable total limiting error per kilometre.

$$\begin{array}{|c|}\hline \tau = U \\ \hline \tau^2 = V^2 - \frac{1}{5}\eta^2 \\ \hline \end{array}$$

with U = limit of u_L, u_F, $u_{F\gamma}$ for L_m, $F_m \geqslant Z$ using

V = limit of v_L for $L_m \geqslant Z$

$$u_L{}^2 = \frac{1}{9} mean \frac{\lambda^2}{L}, \quad v_{L^2} = \frac{1}{9} mean \frac{\mu^2}{L}, \quad u_{F^2} = \frac{4}{9} mean \frac{\phi^2}{F}. \quad u^2{}_{F\gamma} = \frac{4}{9u_F} \sum \frac{\gamma^2}{L}$$

u_L, v_L, u_F can be written in the following forms:

$$\begin{cases} u'^2_L = \frac{1}{9n_L} \Sigma \frac{\lambda^2}{L} \\ \text{or} \\ u''^2_L = \frac{1}{9} \frac{\Sigma\lambda^2}{\Sigma L} \end{cases} \quad \begin{cases} v'^2_L = \frac{1}{9n_L} \Sigma \frac{\mu^2}{L} \\ \text{or} \\ v''^2_L = \frac{1}{9} \frac{\Sigma\mu^2}{\Sigma L} \end{cases} \quad \begin{cases} u'^2_F = \frac{4}{9n_F} \Sigma \frac{\phi^2}{F} \\ \text{or} \\ u''^2_F = \frac{4}{9} \frac{\Sigma\phi^2}{\Sigma F} \end{cases} \quad \Bigg| \quad u^2_F = \frac{4}{9n_{F_\gamma}} \Sigma\frac{\gamma^2}{L}$$

The limit Z is reached when the u, v no longer vary appreciably with L_m, F_m. From these an adequate approximation can thus be obtained.

Coefficient η, probable accidental error per kilometre.

$$\boxed{\eta^2 = u^2_R - \zeta^2 j^2} \quad \text{with } u^2_R = \frac{1}{9} \text{ mean } \frac{\rho^2}{R} \text{ and } j^2 = \frac{K}{Z} \text{ mean } R \ (K = 2 \text{ or } 3)$$

$$\text{using for } u_R \text{ and } j \text{ at the same time} \begin{cases} \text{either } u'^2_R = \frac{1}{9n_R} \Sigma\frac{\rho^2}{R} \text{ and } j'^2 = \frac{K}{Z} R_m \\ \text{or} \quad u''^2_R = \frac{1}{9} \frac{\Sigma\rho^2}{\Sigma R} \text{ and } j''^2 = \frac{K}{Z} \frac{\Sigma R^2}{\Sigma R} \end{cases}$$

Coefficient ζ, probable accidental limiting value (per kilometre) of the systematic error.

$$\zeta^2 = U^2 - \eta^2 = V^2 - \frac{6}{5} \eta^2$$

Derived formulas, giving τ, η, ζ.

$$\tau^2 = U^2 = \frac{(1 - j^2) V^2 - \frac{1}{5} u_R^2}{1 - \frac{6}{5} j^2} \qquad \begin{cases} \eta^2 = \frac{u_R^2 - j^2 U^2}{1 - j^2} = \frac{u_R^2 - j^2 V^2}{1 - \frac{6}{5} j^2} \\ \zeta^2 = \frac{U^2 - u_R^2}{1 - j^2} = \frac{V^2 - \frac{6}{5} u_R^2}{1 - \frac{6}{5} j^2} \end{cases}$$

Remarks - To furnish a reasonable approximation, the means u, v should be evaluated for at least ten elements.

The forms, u', v' yield a somewhat better approximation, especially when the number of elements is small; the forms u'', v'' simplify the computation somewhat.

To compute u_F (u'_F, u''_F) when the limit U_F is attained by the use of elementary circuits (frequent case), it is preferable to take into account the outside circuit of perimeter F_e and closure ϕ_e, and to write:

$$u'^{\,2}_F = \frac{4}{9(n_F + 1)} \left(\frac{\phi^2}{F} + \frac{\phi_e{}^2}{F_e} \right), \quad u''^{\,2}_F = \frac{4}{9(n_F + 1)} \left(n_F \frac{\Sigma\phi^2}{\Sigma F} + \frac{\phi_e{}^2}{F_e} \right)$$

For the calculation of u_F, it is necessary to introduce all the segments L involved in the adjustment, and n_F is the total number of elementary circuits, that is, of independent closure equations.

1.4 Definition of levelling of high precision and precise levelling

By definition, a method of levelling is classified internationally as follows by the value of the coefficient τ, probable total limiting error per kilometre, computed from the above formulas:

levelling of high precision: $\tau \leqslant$ 2 mm.
precise levelling: 2 mm $< \tau \leqslant$ 6 mm.

2. ADVICE CONCERNING THE EXECUTION OF LEVELLING OF HIGH PRECISION

This advice does not apply to those exceptional sights that are necessary to cross natural obstacles such as rivers, etc.

2.1 Instruments

Level - The magnification of the telescope should be at least 25 diameters and if possible should reach 30 or 40 diameters. On the other hand, in order to ensure proper illumination and definition, the aperture of the eyepiece ring should have a diameter of at least 1.5 millimetres.

The radius of curvature of the vial should be kept between 40 and 100 metres, the latter limit not to be exceeded for it is already extreme for observations like levelling made in the open air.

For the highest temperatures at which work is possible, the length of the bubble should never become less than 25 millimetres, since shorter bubbles do not have sufficient mobility.

Rods - If the level is not equipped with a device eliminating the necessity of estimating fractions of the smallest graduations, the smallest division of the rod should be in proportion to the usual length of sight and therefore smaller when the sights are usually shorter. Thus for operations in rough country, where the sights average little more than 20 to 25 metres, the smallest graduation of the rod should be reduced to two millimetres. It should not, on the other hand, exceed one centimetre.

The division should be marked, as far as possible, on invar strips. If this is not done, then in order that it may be possible to determine the variation of the lengths of the wooden rods daily, even in the field, and to take account of it in the computations, compensating rods should be used, in which a single strip of invar might replace the bimetrallic strip (iron and brass) set into the body of the rod (Col. Goulier's model).

To ensure their verticality at the moment of reading, the rods should be furnished with spherical levels of 0.2 metre to 0.5 metre radius of curvature. In addition, it may be convenient, the more easily to hold the rods steady in the vertical position, to provide them with handles and to use one, or better two, props.

2.2 Methods of operation and computation

Operations in the field

1. Each line should be run twice, as far as possible under independent conditions, especially on different days or at different times of the day.

2. Two rods should be used simultaneously.

3. The sights should not be too long. The error, due to refraction and to the unsteadiness of the graduations, increases more rapidly than the length of the sights. They should be regulated, according to atmospheric conditions, so that this error remains consistent with the definition of Levelling of High Precision.

4. At each set-up the level should be placed approximately at equal distances from the rods.

5. The readings used in the computation of the differences in level should not be made on the first lower decimeters of the rod.

6. The operations should be conducted in such a manner as to reduce as much as possible the influence of causes that might involve systematic errors.

Computations

1. The computations should be made in duplicate and, as far as possible, by two different methods.

2. The differences in elevation obtained, forward and backward, should be compared, at least from bench mark to bench mark. The interval of comparison should be chosen small enough to permit, in each case, a good check on the precision.

Parts of the line should be rerun as a verification when the comparison brings out discrepancies exceeding the following tolerances:

Let η and τ be the characteristic coefficients for the accidental and total errors, in mm per km, computed by means of the above formulas, for levelling done by the same method as the levelling under consideration.

The discrepancy should not exceed, in mm, the following values, distances being expressed in km:

for a distance R in the order of a km: $\rho = 6 \text{ to } 8\eta\sqrt{R}$;

for a distance L at least several tens of kms: $\lambda = 6 \text{ to } 8\tau\sqrt{L}$;

for an intermediary distance L, it is necessary to replace in the above formula the coefficient τ by a factor τ_L lying between η and τ, and increasing with L.

Finally it is further necessary to make sure that τ, the probable total limiting error per kilometre, computed for the whole net, does not exceed the limit permitting the method followed to be classed as one of high precision.

Adjustment

1. For the final calculation of the altitudes of bench marks, the corrections should be applied to allow for the lack of parallelism of level surfaces.

2. In the adjustment of the errors of a net, the total probable error to be expected on each link, due to errors of every sort, should be taken into account.

3. In addition to the elevations of bench marks supplied to the public for practical use, it would be interesting from a scientific point of view if the nations would publish the elevations of at least their principal bench marks, particularly those to which tidal observations are referred, according to a system in which the adjustment is made on rational principles, by taking account solely of the levelling operations, and by assuming a priori the elevation of a single fixed point.

As regards the determination of standard elevations for practical purposes, each nation remains free to adjust its level net, in accordance with local conditions, by following the method that seems to it most convenient, and expecially by assuming as zero (or approximately zero) the mean levels of the sea as determined at various points.

Schedule 2

Reference Spheroids and Projections

1. The national datum is the Australian Geodetic Datum (AGD) which consists of:

> Reference Spheroid: The Australian National Spheroid with an equatorial radius of 6 378 160 metres and a flattening of 1/298.25.
>
> Origin: Johnston Geodetic Station situated in the Northern Territory at east longitude 133°12'30.0771" and south latitude 25°56'54.5515" with a ground level height of 571.2 metres above the Australian National Spheroid.
>
> Note: The ground level height of Johnston Geodetic Station above the Australian Height Datum is 566.3 metres.

2. World Geodetic System 1972 (WGS72) shall be the geodetic reference system for all surveys of:

- Lord Howe Island;
- Macquarie Island;
- all Australian External Territories including the Coral Sea Islands Territory and the Territory of Ashmore and Cartier Islands; Cocos, Christmas, Norfolk, Heard and McDonald Islands and the Australian Antarctic Territory;
- any other islands, reefs, etc, which cannot be connected to the AGD by direct measurement; subject to the proviso that surveys falling within the area of any topographic map at 1:250 000 and larger scales in the Australian series shall be on the Australian Geodetic Datum.

The spheroid used in WGS72 has an equatorial radius of 6 378 135 metres and a flattening of 1/298.26.

3. Rectangular coordinates shall be computed from geodetic coordinates on either datum by the Universal Transverse Mercator (UTM) projection. AGD coordinates so converted are called Australian Map Grid (AMG) coordinates - see NMC Special Publication 7, The Australian Map Grid Technical Manual. In Antarctica south of latitude 79°30'S rectangular coordinates shall be obtained from WGS72 coordinates by the Polar Stereographic Grid system.

Schedule 3

RECOMMENDED SURVEY PRACTICES

1. ANGULAR OBSERVATIONS

1.1 First Order Horizontal

1.1.1 One round of observations should consist of a face right pointing to each station in turn proceeding clockwise reversing on the last station, and a face left pointing on each station in turn proceeding anti-clockwise. The means should then be deduced. The closing of the horizon is optional.

1.1.2 One set of horizontal observations should consist of six rounds on six different zeros so chosen as to divide the main circle and the micrometer drum into equal parts.

1.1.3 Zeros could be selected as follows:

1.1.3.1 For Wild T2 Tavistock, DKM Kern or similar instrument

00 00 10
30 11 50
60 03 30
90 15 10
120 05 50
150 18 30

Subsequent sets can be advanced by increments of 10 degrees on each zero.

1.1.3.2 For Wild T3

00 00 05
30 02 15
60 00 25
90 02 35
120 00 45
150 02 55

Subsequent sets can be advanced by increments of 10 degrees on each zero.

1.2 All classifications - Acceptance criteria for horizontal angles

1.2.1 Recommended ranges and standards of accuracy for classifications for horizontal angular observations are shown below in tabulated form. Geodetic instruments being used for special High Precision and First Order angular work, and approved 1" instruments for Second and Third Order work.

1.2.1.1 Special High Precision

As specified by requirements of the project.

1.2.1.2 First, Second and Third Order

	First	Second	Third
Range within each set of 6 zeros should seldom exceed (a)	6"	6"	6"
Range within each set of 6 zeros not to exceed	8	10	12
Residual from mean of any direction should seldom exceed (a)	3	3	3
Residual from mean of any direction not to exceed	4	5	6
Minimum number of sets each of 6 zeros	6(b)	2	1
Range between sets should seldom exceed (a)	2	3	(Not specified)
Range between sets not to exceed	4	4	(Not specified)

Notes:

(a) Where these ranges are exceeded it is essential that additional observations be made.

(b) A minimum of 3 sets should be taken on each of two days.

1.2.1.3 Fourth Order

Accuracy will be directly dependent on scale of mapping.

1.3 Astronomical Observations

1.3.1 Laplace stations should normally be observed with special astronomical theodolites at intervals of 4 to 6 geodetic stations apart but wherever there is evidence of excessive horizontal refraction pairs of Laplace stations should be observed at ends of common azimuth lines.

1.3.1.1 Laplace azimuths should be observed in such a manner that the standard error of the mean of the observations should not exceed 0.4" arc.

1.3.1.2 All observations at Laplace stations are to be reduced in accordance with Division of National Mapping Technical Report 1: "Small corrections to Astronomic Observations".

1.3.2 Provisional specification for fourth order astronomical determinations.

1.3.2.1 There shall be at least four separate determinations for latitude and longitude and the resultant standard error of the mean of these observations shall not exceed ± 3 seconds of arc in each component.

1.4 Vertical Angles

1.4.1 Where it is not possible to determine the elevation of a geodetic survey station by differential levelling, such elevation may be determined by vertical angle observations.

1.4.2 Reciprocal vertical angles should be observed under conditions of minimum refraction. These conditions normally occur between 1400 hours and 1600 hours L.M.T.

1.4.3 Vertical angles should be observed with an approved 1 second or geodetic theodolite.

1.4.4 At least one set of 3 or more pairs (Face Left and Face Right) should be observed at each terminal.

1.4.5 Where the range between the 3 pairs exceeds 8 seconds, sufficient additional pairs should be observed to obtain further comparisons and to improve the accuracy of the set.

1.4.6 Where the line is not included in a closed trigonometrical figure, such as a Tellurometer traverse, two such sets of vertical angles should be observed during the hours prescribed but separated by at least 15 minutes of time.

1.4.7 Within the prescribed conditions it is desirable that reciprocal vertical angles be observed simultaneously. When observations are taken at any other time it is essential they be observed simultaneously.

2. DISTANCE MEASUREMENT

2.1 First Order using radio frequency instruments

2.1.1 Two separate measurements should be made for each line on different days and preferably under different meteorological conditions.

2.1.2 It is desirable, but not essential, that the second day's measurement be effected in the opposite direction.

2.1.3 Measurements shall only be undertaken when atmospheric conditions are such as to ensure homogeneity of the air mass along the length of the line i.e. windy with either complete or zero cloud cover.

2.1.4 When the two measurements differ by more than 1/200 000, additional separate measurements should be made until satisfactory agreement is reached.

2.1.5 Coarse readings must be checked and reduced in the field to show that they are not ambiguous.

2.1.6 A full range of carrier frequencies must be used when taking fine readings. The number of frequencies read depends on the characteristics of the line under measurement.

2.1.7 Should the graph of the ground swing exhibit a cyclic tendency, a full cycle should be completed if possible. Where the graph is of a peculiar shape, the measurement should be repeated from a different stand point.

2.1.8 Meteorological readings must be taken with utmost care.

2.1.8.1 Barometers and thermometers must be calibrated and the index error of barometers must be checked monthly.

2.1.8.2 At each station at least three sets of meteorological readings are to be taken at equal intervals covering the period of the fine readings.

2.1.8.3 Temperatures are to be read or estimated to 0.1°C and pressures read to 0.5 millibar.

2.1.8.4 The vapour pressure must be calculated in the field - any change in the dew point in excess of 1°C or 2 mb of vapour pressure at a station during observation should be suspect. Where vapour pressure readings made at the two stations differ by more than 2 mb the measurement should be rejected.

2.1.8.5 Psychrometers must have a minimum air flow across the thermometer bulbs of 3 metres per second. When the two instruments are set up on the ground the psychrometers must be operated 2 metres above the ground with the thermometer bulbs directed into the wind. They must be shaded, must not be near vegetation and must fulfil the manufacturer's reliability criteria.

2.1.8.6 Thermometers must be of the mercury-in-glass type graduated on the glass in intervals of 0.5°C or smaller.

2.1.8.7 Barometers must be of the type which have a guaranteed sensitivity of at least 0.5 mb.

2.1.8.8 Standard lapse rates for temperature and pressure should be used to test for favourable meteorological conditions and to guard against reading blunders.

2.1.9 All crystal frequencies must be recalibrated by a recognised testing laboratory at least twice a year and before and after repair and the instruments themselves recalibrated.

2.2 First Order using visible and near-visible light instruments

2.2.1 A total of 8 measurements shall be made over two separate days and preferably under different meteorological conditions. On each day two sets separated by at least two hours shall be measured between 11.00 am and 1½ hours before sunset. A set shall consist of two consecutive measurements.

2.2.2 It is not essential that the second day's measurement should be effected in the opposite direction.

2.2.3 Measurements shall only be undertaken when atmospheric conditions are such as to ensure homogeneity of the air mass along the length of the line i.e. windy, with either complete or zero cloud cover.

2.2.4 The least number of prisms consistent with a tuneable return signal shall be used.

2.2.5 When the two acceptable measurements, after the application of meteorological corrections, differ by more than 1:300 000, further measurements should be made until satisfactory agreement is reached. A rule-of-thumb criterion for rejection, where there are three of more measurements, is that if the omission of any one measurement reduces the spread by more than one half, it should be rejected.

2.2.6 Meteorological readings must be taken with the utmost care.

2.2.6.1 Barometers and thermometers must be calibrated and the index error of the barometers must be checked monthly.

2.2.6.2 At each station sets of meteorological readings are to be taken immediately before and after each measurement.

2.2.6.3 Temperatures are to be read or estimated to 0.1°C and pressures read to 0.5 millibar.

2.2.6.4 Psychrometers must have a minimum air flow across the thermometer bulbs of 3 metres per second. When the instrument and reflectors are set up on the ground psychrometers must be operated 2 metres above the ground with the thermometer bulbs directed into the wind. They must be shaded, must not be near vegetation and must fulfil the manufacturer's reliability criteria.

2.2.6.5 Thermometers must be of the mercury-in-glass type graduated on the glass in intervals of 0.5°C or smaller.

2.2.6.6 Barometers must be of the type which have a guaranteed sensitivity of at least 0.5 mb.

2.2.6.7 Standard lapse rates for temperature and pressure should be used to test for favourable meteorological conditions and to guard against reading blunders.

2.2.7 Reciprocal vertical angles are to be observed for lines over 30 km within 15 minutes of a distance measurement to determine the coefficient of refraction.

2.2.8 All modulation frequencies must be recalibrated using a suitable frequency counter at least twice a year and also before and after repair. The instruments themselves must be recalibrated before and after each project. For projects requiring the highest precision modulation frequencies are monitored throughout the measuring program.

3. DOPPLER OBSERVATIONS

3.1 Only those satellites having an elevation of 10^{o} or greater at the time of closest approach should be used in the computation.

3.2 For most accurate results the antenna should not be raised above ground level more than the height of an instrument tripod and should be sited so that there are no obstructions above 10^{o} elevation.

3.3 The antenna should not be sited near metal objects such as towers, wire fences, radio aerials or motor vehicles. If local interference is detected a new site should be selected.

3.4 The antenna should be inspected regularly to ensure that it has not been disturbed by stock, birds or other wildlife.

3.5 Sufficient time should be allowed to ensure oscillator stability and once set up the receiver should not be moved until observations have been completed.

3.6 Pressure, wet and dry temperatures and weather descriptions should be recorded at every opportunity.

3.7 Cassettes should be used once only.

4. DIFFERENTIAL LEVELLING

4.1 First Order

4.1.1 Instruments

4.1.1.1 An approved auto collimating or precision spirit level with parallel plate refractor should be used for first order levelling.

4.1.1.2 Recognised invar staves or precision folding staves should be used. Staves should be numbered and calibrated before use and re-calibrated at approximately three monthly intervals when in use.

4.1.2 Levelling Practice

4.1.2.1 Setting up the level

The level should always be set up to ensure maximum stability during observations. When using precision spirit level it should at all times be shaded.

When using automatic levels the unsystematic procedure of centring the circular bubble is to be adhered to. That means if instrument stations in a levelling run are numbered in sequence the circular bubble is to be centred at:

(a) odd numbered stations with telescope pointing in the direction of the backsight and at

(b) even numbered stations with telescope pointing in the direction of the foresight or vice versa

Prior to the first reading at any instrument station the telescope is to be turned slightly first in one direction then the other.

.1.2.2 Use of Staves

Differential levelling should be observed by use of two staves. Bases of staves should be inspected and cleaned if necessary at every change point. At each change point the staff should be placed on a metal ground plate firmly set so that no settlement can occur during the period of time taken for the required backsight and foresight. Metal spikes driven firmly and securely into the ground may be used in lieu of ground plates. When observations are being made the staff shall be held vertical by reference to the circular bubble which should be checked each day for verticality. This test should be made indoors to avoid sun and wind effects.

4.1.2.3 Collimation

At least once during each day's work a test for error in collimation of the level should be made and recorded in the level book. Collimation error should be adjusted where it exceeds 0.002 m in a distance of 50 metres. The collimation error should be observed and recorded after adjustment.

4.1.2.4 Length of Sight

The length of any levelling sight should be such as to permit the certain reading of the staff to the required accuracy and should not exceed 40 metres. Backsight and foresight should be equal in length.

Where terrain difficulties do not permit equal sights an explanatory note relevant to the entries of distances should be made in the level book. Efforts should be made to equalise total backsight and foresight distances during the last few observations between permanent bench marks. The total length of backsights should in no case differ from the total length of foresights by more than 15 metres.

4.1.2.5 Refraction

In order to avoid some of the effect of abnormal refraction due to a line of sight grazing the ground surface, no sight line between level and staff shall be less than 0.3 metre above ground surface.

4.1.2.6 Readings

4.1.2.6.1 When using metric staves, observations and recordings of the centre wire only should be made to 0.0003 metre either by use of a parallel plate refractor or by three readings through a plain telescope.

4.1.2.7 Procedure

4.1.2.7.1 The second levelling of a section should proceed in the reverse direction to the first levelling and should preferably be performed by a different survey party equipped with a second complete set of instruments.

4.1.2.7.2 For all levelling, the foresight staff must remain at the change point, and the same staff used for the next backsight.

4.1.2.8 Accuracy

4.1.2.8.1 The two levellings of each section between permanent bench marks shall not differ by more than $4\sqrt{K}$ mm where K is the distance in kilometres between bench marks measured along the levelling route. Circuit enclosures shall not exceed this same limit, where K is the length of the circuit in kilometres along the levelling route.

4.1.2.9 Records

4.1.2.9.1 All recording of observations in level books should be made in ink. The level book should record each day; the date, the collimation test, the details of the section of levelling, the names of the personnel in the survey party, the serial numbers of the instruments and staves, the observing conditions, the air temperature observed at commencement, estimated middle and end of day's operations, and the time of beginning and ending of each flight of levels between bench marks.

4.2 Third Order Levelling

4.2.1 Instruments

4.2.1.1 A suitable auto collimating or spirit level should be used for THIRD ORDER LEVELLING.

4.2.1.2 Staves constructed of a suitable material should be used. They should be numbered and calibrated before use and preferably re-calibrated at six monthly intervals.

4.2.2 Levelling Practice

4.2.2.1 Setting up the level -

The level should always be set up to ensure maximum stability during observations. When using a spirit level it is desirable that it be shaded. When using automatic levels the unsystematic procedure of centering the circular bubble is to be adhered to. That means if instrument stations in a levelling run are numbered in sequence the circular bubble is to be centred at:

(a) odd numbered stations with telescope pointing in the direction of the backsight and at

(b) even numbered stations with telescope pointing in the direction of the foresight or vice versa.

Prior to the first reading at any instrument station the telescope is to be turned slightly first in one direction then the other.

4.2.2.2 Use of Staves

It is considered that the use of two staves is good practice. Bases of staves should be inspected and cleaned if necessary at every change point. Change points should be so selected as to ensure stability of the staff during observations.

When observations are being made the staff shall be held vertical by reference to a circular bubble which should be checked periodically for verticality.

4.2.2.3 Collimation

A test for collimation error should be made preferably each day but not less than every third day and recorded in the level book. Collimation error should be adjusted where it exceeds 0.002 m in a distance of 50 metres. The collimation error should be observed and recorded after adjustment.

4.2.2.4 Length of Sight

The length of any levelling sight should be such as to permit the certain reading of the staff to the required accuracy and should not exceed 90 metres. Backsight and foresight should be preferably equal in length. Where terrain difficulties do not permit equal sights efforts should be made to equalise total backsight and foresight distances during the last few observations between permanent bench marks. Between bench marks the total length of backsights should not differ from the total length of foresights by more than 45 metres.

4 2.2.5 Refraction

In order to diminish the effect of abnormal refraction due to a line of sight grazing the ground surface, no line of sight between level and staff should be less than 0.3 metre above the ground surface.

.2.2.6 Readings

Staves should be read and recorded to 0.001 metre.

4 2.2.7 Procedure

4 2.2.7.1 All sections must be check levelled preferably by a second survey party.

.2.2.7.2 For all levelling the foresight staff must remain at the change point, and the same staff used for the next backsight.

4.2.2.8 Accuracy

The two levellings of each section between permanent bench marks shall not differ by more than $12\sqrt{K}$ mm where K is the distance in kilometres between bench marks measured along the levelling route. Circuit closures shall not exceed this same limit, where K is the length of the circuit in kilometres along the levelling route.

4.2.3 Records

4.2.3.1 All recording of observations in level books must be made in ink. The level book should record each day: the date, the collimation test, the details of the section of levelling, the names of the personnel in the survey party, the serial numbers of the instrument and staves, the observing conditions and the time of beginning and ending of each flight of levels between bench marks.

4.3 Fourth Order Levelling

4.3.1 Instruments

4.3.1.1 A suitable auto collimating or spirit level should be used for fourth order levelling.

4.3.1.2 Wooden staves should preferably be used. If metal staves, other than invar, are used the temperature should be recorded at intervals sufficient to ensure proper adjustment of staff length.

4.3.2 Levelling practice

4.3.2.1 Setting up the level

The level should always be set up to ensure maximum stability during observations. When using a spirit level the unsystematic procedure of centring the circular bubble is to be adhered to. That means that if instrument stations in a levelling run are numbered in sequence the circular bubble is to be centred at:

(a) odd numbered stations with the telescope pointing in the direction of the backsight and at

(b) even numbered stations with the telescope pointing in the direction of the foresight.

Prior to the first reading at any instrument station the telescope is to be turned slightly first in one direction then the other.

4.3.2.2 Use of Staves

It is considered that the use of two staves is good practice. Bases of staves should be inspected and cleaned if necessary at every change point. Change points should be so selected as to ensure stability of the staff during observations.

When observations are being made the staff shall be held vertical by reference to a circular bubble which should be checked periodically for adjustment. Except for foot-metric levelling all staves should be graduated in metres.

4.3.2.3 Collimation

A test for collimation error should be made each day and recorded in the level book. Collimation error should be adjusted where it exceeds 10 seconds of arc. The collimation error should be observed and recorded after adjustment.

4.3.2.4 Length of sight

The length of any levelling sight should be such as to permit the certain reading of the staff to the required accuracy and should not exceed 200 metres. Backsights and foresights should preferably be of equal length. Where terrain difficulties do not permit equal sights efforts should be made to equalise total backsight and foresight distances during the last few observations between bench marks. Between bench marks the total length of backsights should not differ from the total length of foresights by more than 100 metres.

4.3.2.5 Refraction

In order to diminish the effect of abnormal refraction due to a line of sight grazing the ground or other surfaces no line of sight should be less than 0.5 metre above the ground or any other intervening surface.

4.3.2.6 Readings

Metric staves should be observed and recorded to 0.005 metre. When foot-metric levelling is being undertaken the foot face of staves should be observed and recorded to 0.01 foot.

4.3.2.7 Accuracy

If the levelling is two-way the two levellings of each section between permanent bench marks shall not differ by more than $18\sqrt{K}$ mm where K is the distance in kilometres between bench marks measured along the levelling route. If the levelling is one-way foot-metric using dual faced levelling staves the two levellings of a section, one in metres and the other in feet and converted to metres, shall not differ by more than this same limit. Circuit closures, where circuits are made up wholly of 4th order levelling, shall not exceed this same limit, where K is the length of the circuit in kilometres along the levelling route. Where levelling is one-way, either foot-metric or metric only, the extremities of the traverses will be connected to traverses of a higher order and these circuit closures shall not exceed $36\sqrt{K}$ mm where K is the length of the circuit in kilometres along the levelling route.

4.3.3 Records

All recordings of observations in level books must be made in ink or ball point pen. The level book should record each day: the date, the collimation test, the details of the section of levelling, the names of the personnel of the survey party, the serial numbers of the instrument and staves, the observing conditions and the marks levelled to.

4.3.4 Marks

All permanent bench marks established along 4th order level traverses shall be constructed in accordance with these specifications and recommended practices.

Appendix 1 to
Schedule 3

Levelling Staff Calibration*

(a) Determination of the Overall Length of Precision Staffs

When a precision staff is initially calibrated by an appropriate authority it will be necessary for the true length of one nominated interval, preferably the overall length or close thereto, to be quoted.

This nominated interval should be remeasured from time to time during field measurements to ensure that no significant change has taken place. Any change should not exceed say, 0.2 mm/metre if the requirements for Precision Levelling are to be continued to be met.

While it would be fundamentally better to remeasure the nominated interval on the staff as a single entity there are a number of practical difficulties in carrying this out in the field.

A determination that entails the measurement of a number of contiguous sub-intervals (say three) of the nominated interval should be satisfactory however, provided an accuracy of 0.1 mm/metre is achieved. A 1 m scale, preferably of invar, bevel-edged and divided at intervals of 10 mm throughout with the first and last 10 mm subdivided to 0.5 mm, and used with a magnifier of a power at least x 10, should be adequate for this purpose; the graduation lines should be 0.05 to 0.1 mm wide, and of uniform width throughout. Account should of course be taken of the temperature during measurements.

If the field measurements indicate that the nominated interval has changed in excess of 0.2 mm/metre it would of course, be possible to proceed with an adjusted value of the "mean staff interval" on the assumption that the change has occurred reasonably uniformly along the staff.

(b) Calibration of 3rd Order Staffs

In considering 3rd order staffs the working group decided to look into both initial calibration and subsequent field checks.

Initial Calibration

Whilst, on theoretical grounds, a calibration in accordance with the procedure described by Thwaite (1962), based on a random selection of intervals is safer, practical considerations and experience suggest that a method which requires simpler equipment may well meet the calibration needs of 3rd order staffs.

A number of lengths (say ten) are to be measured on each section of the staff using a 1 m bevelled edged scale, divided throughout to 10 mm, with the first and last 10 mm subdivided to 0.5 mm, and a magnifier (x10). The intervals measured should not be systematic in length and should include long and short intervals. The staff may be vertical or horizontal.

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* These recommendations are based on the Report of a Technical Subcommittee to the National Mapping Council.

The estimate of the "staff metre" is to be based on these measurements alone. As these measurements do not include any joints the errors in the joints should be of the same order as those encountered in the staff subdivisions, neither very much greater or smaller.

A nominated interval on each section of the staff, together with a small interval bridging each joint, should be measured for comparison with subsequent field checks.

Particular attention must be paid to the joints in the staff. Measurement of a small interval bridging a joint can be readily measured on a folding staff with a short bevel-edged scale; for telescopic staffs a scale with adjustable off-set should be used (see accompanying figure). It is important that all measurements across the joins be made with the staff vertical.

An observational standard deviation for the measurements of 0.3 mm, with a systematic error not exceeding ± 0.2 mm/metre should be achieved.

The "staff metre" is assessed from the sum of measured lengths of the intervals, excluding measurements over the joints, divided by the sum of the normal lengths. Using this value the evenness of subdivision is obtained by comparison with the measured intervals and a standard error of graduation is computed.

Field Checks

As in the case of precision staffs the field checks are intended to ensure that no significant change has taken place.

Each nominated interval should be measured from time to time using a bevelled edge scale and magnifier similar to those described for use in field checks on precision scales. The accuracy of measurement should be ± 0.2 mm/metre or better and if there is a change in the remeasured interval, from its initial calibration, in excess of say 0.4 mm/metre an adjustment to the estimate of the "staff metre" should be made or the staff should be withdrawn from use.

General

The various types of staffs available, and in use throughout Australia, indicate that the initial purchase is not governed primarily by the needs of control levelling circuits.

A purchaser would necessarily choose a staff suited to the needs of his continuing projects and, if at all possible, it would be an advantage to use these staffs on control level networks. An engineering establishment might choose telescopic staffs of 5 m length as basic equipment.

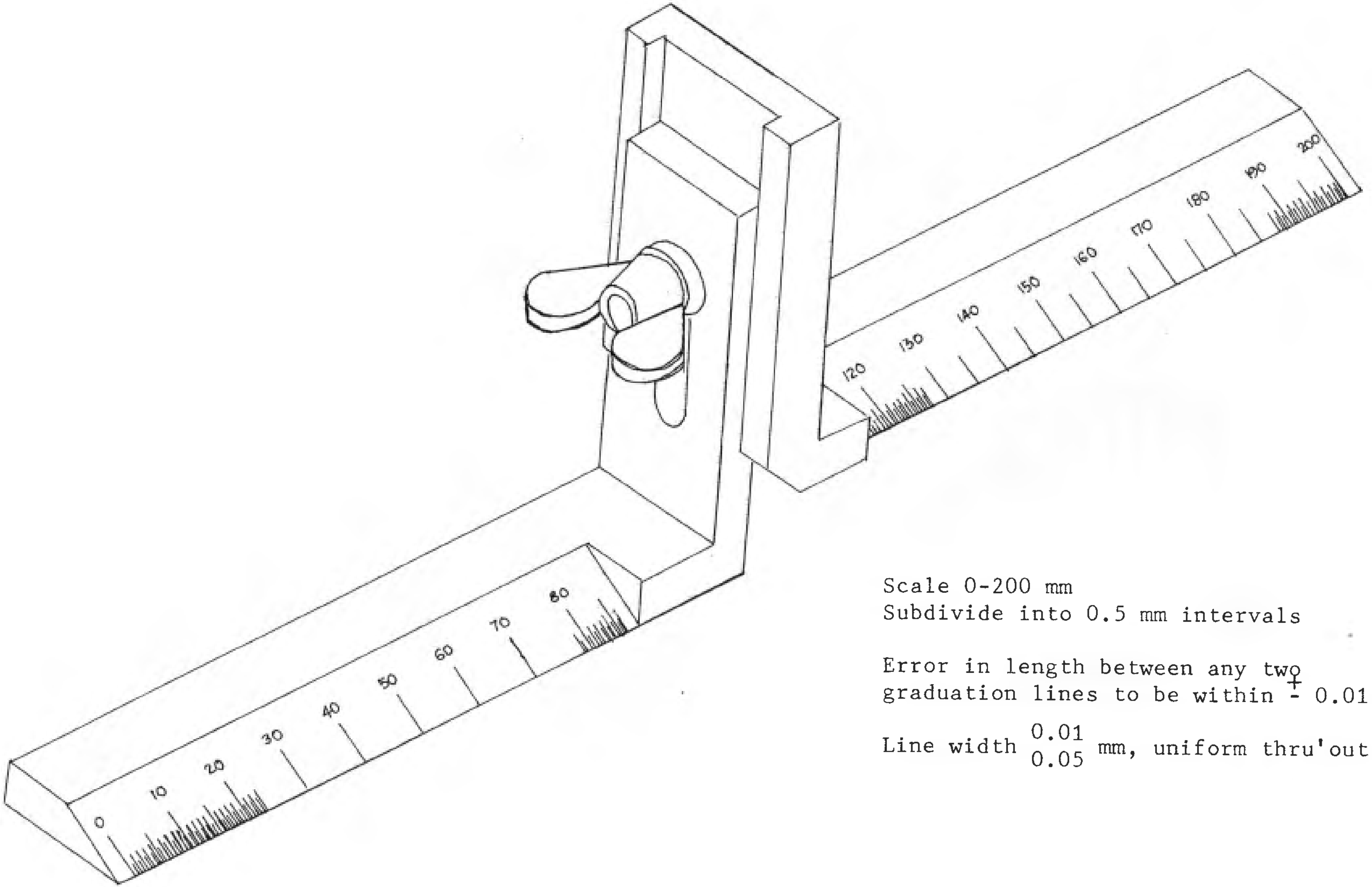
The major factor in purchasing utility staffs by the various survey agencies is obviously ease of transport. Once telescopic or folding staffs are adopted, then the user is committed to regular precautionary inspection of inherent faults, ie the joint hinge, clamp lock or slide.

While temperature corrections can be applied to staffs of metal construction, the combined temperature and humidity effect on wooden or plastic staffs is completely ignored in survey practice. Although the working group has not investigated this field, it is apparent that a wooden staff with its enamel coating worn or chipped must be more prone to humidity change than a staff in its original condition.

It is therefore, necessary to examine staffs, in use for third order levelling, for general wear as well as to ensure that the overall length meets the necessary standards.

For levelling of higher precision than third order, special type staffs of invar, and unbroken, are recommended.

Thwaite, E.G. (1962) - "Accuracy of Staff Calibration to meet National Mapping Requirements". C.S.I.R.O. National Standards Laboratory, Division of Applied Physics, Interim Report, M.I.R. 1130.



Scale 0-200 mm
Subdivide into 0.5 mm intervals

Error in length between any two graduation lines to be within ± 0.01 mm

Line width $\substack{0.01\\0.05}$ mm, uniform thru'out

METRIC SCALE FOR JOINT MEASUREMENTS OF SURVEYING STAFFS

Schedule 4

RECOMMENDED MARKING PRACTICES

1. STATION MARKING PRACTICES

1.1 Standard station marks and bench marks

1.1.1 All first order triangulation stations, all second and third order triangulation or trilateration stations and selected traverse stations shall be permanently marked and permanent bench marks shall be established along lines of first, second, third and fourth order levelling at intervals apart of one kilometre in densely developed areas and up to about eight kilometres in sparsely developed areas. The basis of selection of traverse stations for permanent marking shall be:

1.1.1.1 Where the adjoining stations are more than eight kilometres apart all stations will be marked.

1.1.1.2 Where adjoining stations are closer than eight kilometres apart, certain stations from eight to sixteen kilometres apart shall be selected for permanent marking.

1.1.2 Permanently marked horizontal control stations and permanent bench marks shall be marked by metal tablets, rods or pipes. These markers shall be made of corrosion resistant material and be firmly set in a pre-cast concrete block or in concrete poured into a roughly cut hole and tamped. The upper surface of the concrete block shall have an area of not less than 160 square centimetres and the lower not less than 300 square centimetres. Unless on a solid base the concrete block shall be 60 centimetres in depth or of such greater depth as is necessary to obtain a sound foundation. The concrete shall be of a good standard quality and the whole shall be constructed and placed in a workmanlike manner. Every standard station mark shall be so constructed and placed as to be capable of serving the purpose of theodolite reference mark and bench mark. To facilitate the use and ease of recovery of horizontal control stations beacons of pattern approved by the survey organisation concerned should be erected over each first and second order horizontal control mark.

1.1.3 Fundamental bench marks should be established at intervals of 40 to 60 kilometres apart. These should be set in solid rock and should be standard bench mark firmly set in a substantial sub-surface concrete block. These bench marks should be protected by a fence or low wall and should preferably be established in position at least one year prior to the survey.

1.1.4 In all third order levelling some form of more permanent bench mark shall be established at intervals of about 40 kilometres and in no case at an interval of more than 160 kilometres. Where possible, it should consist of three standard bench marks, between 30 and 100 metres apart, set in solid rock. Where no solid rock is available, the more permanent mark should consist of three deep bench marks, going down at least 3 metres if the nature of the ground allows.

1.2 Subsidiary bench marks

1.2.1 As many subsidiary bench marks as are practical and economical should be established between standard bench marks. These can be located on such objects as trees, pegs of lasting material, buildings, bridges or other structures of a reasonable permanent nature.

1.3 Special marks

1.3.1 Under circumstances in which it is not practicable to place permanent marks as detailed under 1.1.2, the most suitable mark applicable to the circumstances may be used.

1.4 Sub-surface mark

1.4.1 Sub-surface marks are desirable, at first, second and third order horizontal control stations. Sub-surface marks should preferably be made of concrete not less than 150 mm thick and 250 mm in diameter with the station point marked by a metal tablet, copper bolt, or other durable substance.

The sub-surface mark should be 100 to 125 mm below the base of the concrete surrounding the standard permanent mark and extreme care must be taken that the sub-surface mark is directly underneath the centre of the surface mark.

1.5 Reference marks

1.5.1 At least two, and preferably three reference marks should be set at each first, second and third order horizontal control station. They should be set in concrete and consist of metal markers similar to the standard station mark and preferably inscribed with an arrow pointing toward the station. Particular care should be taken to select sites for these reference marks where they will not be subject to disturbance. Where possible two suitable reference trees or tall stumps should also be marked.

1.6 Azimuth marks

1.6.1 At triangulation stations and at permanent marked traverse stations where a reference azimuth cannot conveniently be obtained by sighting to an adjacent station, azimuths shall be specially observed to suitable reference marks of a permanent nature. These azimuths shall be observed with an accuracy represented by a probable error not in excess of 5 seconds of arc in the case of first and second order stations and not in excess of 10 seconds of arc in the case of third order stations.

2. STATION NAMES AND REFERENCES

2.1 It is recommended that triangulation and, where desired, first order traverse stations shall be referred to by name, other permanently marked traverse stations by a combination of letters and numbers and permanently marked bench marks by the letters BM followed by a number which shall not be repeated within selected subdivisions of a State or Territory.

2.2 Names for triangulation stations should have a geographic significance wherever possible. Care should be taken by the officer in charge of the survey party to ascertain the name which is most prevalent for a particular geographic feature. The Surveyor General or Director of Mapping of the State or Territory is to be accepted as the final authority for these names and reference letters and numbers.

3. DESCRIPTION OF STATIONS

3.1 A clear concise and complete description and a sketch plan shall be made out for each standard mark and for each bench mark established, showing sufficient information to permit the ready location of the mark itself. This report and sketch plan should be filed in convenient form in the central office of the organisation responsible for surveys in the State or Territory.

4. RECOVERY OF ANY STANDARD MARK OF BENCH MARK

4.1 It is recommended that survey organisations engaged in surveys and mapping should instruct their field officers to report on the conditions of standard station marks or bench marks which may be visited by them. If the mark is found, the recovery report should state the condition in which it was found and should give any modifications or additions to the descriptions which would make it more easily found in the future. If the mark is not found, the report should indicate the thoroughness of the search made and give recommendations as to whether or not the station should be marked "lost" in the records. These recovery reports should be filed in the central office responsible for maintaining the records of station descriptions.

4.2 If a standard station mark of a third or higher order of accuracy is found in poor condition by an approved survey organisation and if its proper location can be determined with certainty and the required accuracy, either by a recovered underground mark or by measurements from two or more reference marks the survey organisation should re-mark the station if practicable. If the marker of the original station is recovered, it should be re-set. If an underground mark exists due care must be exercised to ensure that the new surface mark is exactly centred over the sub-surface mark.

4.3 If a bench mark is reported as having been disturbed, an approved survey organisation may establish a new bench mark in the immediate vicinity and determine the level of this new mark from adjacent bench marks using the same order of the original bench mark. It is important that a completely new number be allocated to the new bench mark.

Schedule 5

RECOMMENDED DOCUMENTATION PRACTICES

1. HORIZONTAL CONTROL

1.1 All permanently marked horizontal control stations of potential use for topographic mapping or from which further geodetic surveys can be controlled shall be recorded on a summary form as soon as practicable after the station is established. The forms approved by the National Mapping Council are:

Horizontal Control Station Summary	Annex A
Astronomic Station Summary	Annex B

1.2 Horizontal control station summaries shall be prepared in accordance with the procedure approved by the National Mapping Council and described in Annex C.

1.2.1 The responsibility for the preparation of a final horizontal control station summary lies -

- in a State, with the State organisation which is responsible for the geodetic survey of that State;
- in a Territory, with the Commonwealth organisation which is responsible for the geodetic survey of that Territory.

1.3 Amendments or alterations to the horizontal control station summaries shall be made in accordance with the procedures described in Annex D.

1.4 All horizontal control stations shall, where practicable, be identified on an air photograph and details shall be entered on the station summary. The procedure to be followed is laid down in Annex C, paragraph 2.12.

1.5 Authorities requiring air photo identification of horizontal control stations shall obtain them from the authority which holds the original photos.

1.6 All horizontal control stations, including Laplace and Doppler stations, shall, as soon as possible after establishment, be recorded on a 1:250 000 or larger scale overlay in accordance with the example attached as Annex F. Overlays may be drawn by hand or by machine.

2. VERTICAL CONTROL

2.1 Bench mark sketch plans produced by the various survey authorities shall constitute vertical control station summaries.

2.2 All first, second, third and fourth order level traverses shall be recorded on an overlay to the 1:1 000 000 International Map of the World series and delineation shall be by lines only with numbers of junction points in accordance with the example attached as Annex G.

2.3 All first, second, third and fourth order bench marks shall be recorded on a 1:250 000 or larger scale overlay in accordance with the example attached as Annex F.

2.4 All bench marks shall, where practicable, be identified on air photographs and details shall be entered on the bench mark sketch plans in a similar manner to that described in Annex C, paragraph 2.12.

3. NATIONAL DATA BASE FOR HORIZONTAL AND VERTICAL CONTROL

3.1 The National Mapping Council in Resolution 382 resolved that:

- All members contribute details of horizontal and vertical control stations of geodetic value to the National Geodetic Data Base maintained by the Division of National Mapping; and that
- individual members wishing to introduce digital data bases for all horizontal and vertical control in their State take into account the system adopted for the National Geodetic Data Base and ensure compatibility with it.

3.2 A suite of programs to input, manipulate, output and update the National Geodetic Data Base can be obtained from the Division of National Mapping.

3.3 The following outputs are some examples which may be produced from the National Geodetic Data Base:

1. Information Summary: Annex H
2. Stations within a nominated map area: Annex I
3. Stations within a nominated adjustment: Annex J

NMP/80/190.1

NATIONAL MAPPING COUNCIL OF AUSTRALIA
STATION SUMMARY

Serial No

Authority DIVISION OF NATIONAL MAPPING

Station Number and Name: NM/B/39 BUTLER

Order: FIRST

Original Station Established by: Division of National Mapping Date: 1960

Existing Station Marked by: Division of National Mapping Date. 1962

Reference Books: NM 1984, 1301, 2000, 2087, 2088. 2089, 2090, 2091, 2217, 2353, NM 2354, 2355, 2356, 10903,

Cadastral Location: State Qld County/District

Parish/Hundred Allotment/Section/Portion

Map Name: Canterbury Map Number: SG 54-7 Scale 1: 250,000

DATUM: Australian Geodetic Datum 1966

RECTANGULAR COORDINATES: Australian Map Grid: In Metres

GRID BEARING = ADJ AZIMUTH + CONVERGENCE

AHD HEIGHT 298.2

HEIGHTS: In Metres on the Australian Height Datum

BUTLER NM B 39 SECTION 10 15 SERIAL 13

SOUTH LATITUDE	EAST LONGITUDE	ZONE	EASTING	NORTHING	CONVERGENCE
25. 25. 32.7466	141. 47. 27.0962	54.	579530.143	7187661.380	+0. 20. 22.44

TO		SERIAL	ADJ AZIMUTH	ADJ LENGTH
COLLINS	NM B 101	12	255. 27. 54.39	29693.358
	NM B 40	14	15. 34. 46.98	17867.762

SECTION 15 18 SERIAL 1

NM B	41	3	314 11	9.31	26315.427
NM B	42	4	336 30	5.27	56513.941

SECTION 17 15 SERIAL 10

NM B	38	6	135 30	51.72	28434.995

Access and Locality Sketch:

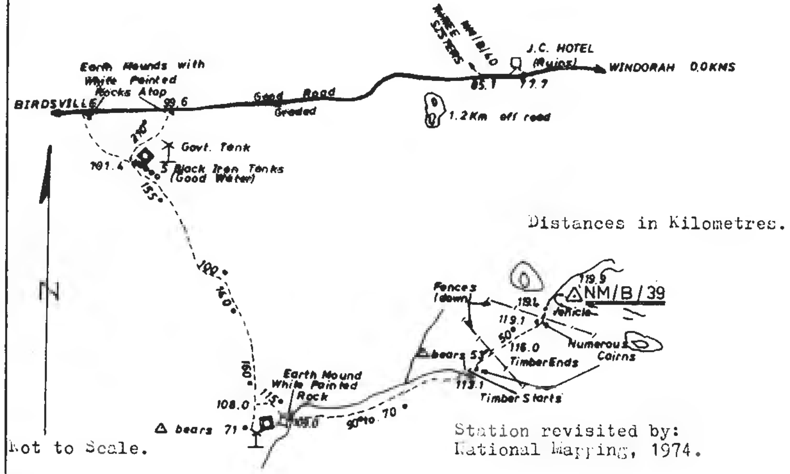
Particulars of station marking and beacon:

Station Mark : 13mm copper tube set in concrete.

Beacon : 3.35m x 10cm x 10cm oregon pole with four 0.91m x 0.61m bondwood vanes set 0.06m below top, erected in centre of 2.4m diam. x 1.8m high rock cairn.
In Oct. 1974, the cairn was found in good condition but the pole and vanes need replacing

Reference Marks: Three 13mm copper tubes set in concrete.

Access : The trig is situated on the western end of a triangular shaped flat top about 4.8kms South of the Windorah - Birdsville road. It could be approached from the North but the more circuitous route shown would appear to give an easier and closer approach. Turn off Windorah - Birdsville road at 99.6kms and follow station track as shown to 108.0kms. From here follow up creek to 113.1kms, then through scattered, and mostly dead timber to 116.0kms. Thence open going to within about 0.8km of vehicle stop. The last section is through rather rougher timber and gutters. There is a short steep climb to trig site.



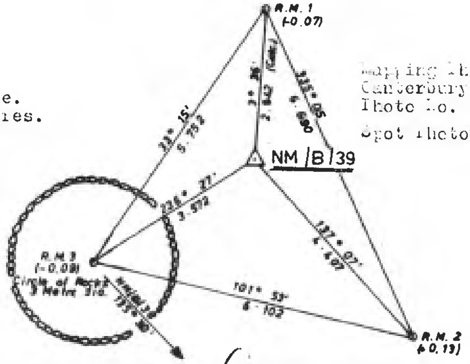
Distances in Kilometres.

Not to Scale.

Station revisited by:
National Mapping, 1974.

Cadastral Connections & Reference Marks

Bearings are true.
Distances in metres.



Mapping Photography:
Canterbury C.B 9. Run 6,
Photo No. 5108.

Spot Photo: C.B 3455 Exp 30-35.(S[illegible])

Photo Identification:

Certified free of transcription errors: S Abod Date: 12-5-75

Approved by: K Leppert Date: 21-8-75

AHQ/DET/1094

Annex A

NMP/80/190.2

NATIONAL MAPPING COUNCIL OF AUSTRALIA
ASTRONOMIC STATION SUMMARY

OBSERVING AUTHORITY DIVISION OF NATIONAL MAPPING

Observer: S. BENNETT Instrument: DKM 3A State: VICTORIA

Recorder: D. GRAY Number: 117347 Sheet No and Name: I 54 - 16 SWAN HILL

Height of Station: 102.4 m Station: BLUE HILLS

17 Height of RO: 92.0 m RO: LIANIDUCK

17 Height of RO: RO:

LATITUDE

Observing Method:	CIRCUM MERIDIAN ALTITUDES
Dates of Observations:	25, 26 May 1976
Standard Deviation of 16 pairs $=\sqrt{\Sigma v^2/(n-1)}$	± 0." 51
Standard Error of Mean $=\sqrt{\Sigma v^2/n(n-1)}$	± 0." 19

LONGITUDE

Observing Method:	35 IMPERSONAL ALMUCANTAR
Dates of Observations:	26, 27 May 1976
Standard Deviation of 16 pairs $=\sqrt{\Sigma v^2/(n-1)}$	± 0." 06
Standard Error of Mean $=\sqrt{\Sigma v^2/n(n-1)}$	± 0." 02

AZIMUTH

Observer: S. BENNETT
Instrument: DKM 3A 117347

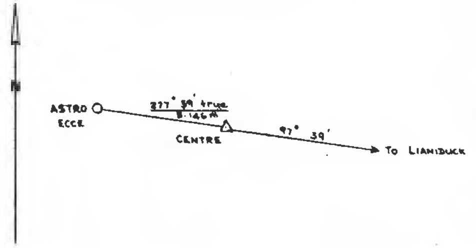
Observing Method:	SIGMA OCTANTIS
Dates of Observations:	25, 26 May 1976
Standard Deviation of 22 zeros $=\sqrt{\Sigma v^2/(n-1)}$	± 1." 27
Standard Error of Mean $=\sqrt{\Sigma v^2/n(n-1)}$	± 0." 27

TIME SIGNALS

Chronometer:	LABTRONICS TYPE 674 No. 305
Chronograph:	FAVAG 600373
Time Signal Origin:	VNG LYNDHURST
Quality of Reception:	GOOD

ECCENTRICITY DIAGRAM

Field Book Numbers:
Latitude and Longitude F.B. No. 15648
Azimuth F.B. No. 15575



LATITUDE CORRECTIONS

		Preliminary Latitude (+N, −S) (DUT 1 included)	−	35	13	07.511
1	+	Eccentricity Correction	−			0.022
2	+	Correction to CIO Pole, $\Delta\phi_P$	+			0.289
3	+	Correction to Sea Level, $\Delta\phi_H$	+			0.016
		FK4 ASTRONOMIC LATITUDE (+N, −S), ϕ_A	−	35	13	07.228

LONGITUDE CORRECTIONS (all in Time)

		Preliminary Longitude (+W, −E) (DUT 1 included)	−	9	30	16.474
4	+	Eccentricity Correction in Time	−			0.013
5	+	$\Delta t = (t_m - t_o)$ in UT2, t_m = Emission Time				0.000
6	−	ΔT_S = UT2-UT1, (ΔT_S =)				0.000
7	−	$\Delta\lambda_I$, Correction to CIO Pole, ($\Delta\lambda_I$ = −.016)	+			0.016
8	+	TF, Transmission Time to Field Station	+			0.004
9	+	Lag in Audio Filter	+			——
10	+	Personal Equation (Almucantars only)	−			——
11	+	Diurnal Aberration (Almucantars only)	−			0.012
		FK4- GREENWICH ASTRONOMIC LONG. IN TIME (+W, −E)	−	9	30	16.479
		FK4- GREENWICH ASTRONOMIC LONG. IN ARC (+W, −E) λ_A	−	142	34	07.185

AZIMUTH CORRECTIONS

		Reference Object	LIANIDUCK						
		Preliminary Azimuth (DUT 1 included)	97	39	30.60				
12	+	Eccentricity Correction			0.00				
13	+	Convergence			0.00				
14	+	Correction to CIO Pole, ΔA_P	+		0.42				
15	+	Correction for Diurnal Aberration	−		0.32				
16	+	Correction for Skew Normals			0.00				
		ASTRONOMIC AZIMUTH, A_A	97	39	30.70				

LAPLACE CORRECTION & DEVIATION OF VERTICAL

Geodetic Datum: AUSTRALIAN NATIONAL SPHEROID

Origin; Station: JOHNSTON GEODETIC STATION

Latitude (+N, −S)	− 25	56	54.5515
Longitude (+W, −E)	− 133	12	30.0771
Geodetic Latitude (+N, −S), ϕ_G	− 35	13	10.58
Geodetic Longitude (+W, −E), λ_G	− 142	34	08.02
$\xi = \phi_A - \phi_G$	+		3.35
$\lambda_A - \lambda_G$	+		0.83
$\eta = (\lambda_A - \lambda_G)\cos\phi$	+		0.68
$\zeta = \xi \sin A + \eta \cos A$	+		3.23

Reference Object: LIANIDUCK

Elevation Angle to RO, E ±	− 0	04	53.00
Astronomic Azimuth, A_A	97	39	30.70
$+(-\zeta \tan E)$ ±			0.00
$+(\lambda_A - \lambda_G)\sin\phi$ ±	−		0.47
LAPLACE AZIMUTH	97	39	30.23

Reference Object:

Elevation Angle to RO, E ±			
Astronomic Azimuth, A_A			
$+(-\zeta \tan E)$ ±			
$+(\lambda_A - \lambda_G)\sin\phi$ ±			
LAPLACE AZIMUTH			

Computed by:

Approved by:

Date: 1/12/76

Note: See back of Summary for reference to correction numbers.

Annex B

FORMULAE FOR CORRECTIONS

Note: The International convention of λ = Longitude (+West, − East) and φ = Latitude (+North, − South) has been adopted. All corrections are described in "Small Corrections to Astronomic Observations" published in the Australian Surveyor, September 1964, Vol. 20, No. 3, pp 199–211, to which paragraph numbers in the right hand column refer.

LATITUDE

1 Eccentricity Correction: If applicable. Diagram must be drawn in relevant space. 10

2 Correction to BIH Pole: $\Delta\varphi_P = -(x \cos\lambda + y \sin\lambda)$, where x and y are interpolated from Circular D published by Bureau International de l'Heure (B.I.H.)

3 Correction to Sea Level: $\Delta\varphi_H = (-170.6'' \, h \sin 2\varphi) 10^{-6}$, where h = metres above sea level. 15

LONGITUDE

4 Eccentricity Correction: Apply in time if applicable. 10

5 $\Delta t = UT_1 - UTC$. Published in Circular D by B.I.H.

6 Correction to BIH Pole: $\Delta\lambda_f = \tan\varphi \, (x \sin\lambda - y \cos\lambda) / 15$.

7 TF, Transmission Time to Field Station: For short wave signals (VNG, WWVH, etc), calculate using Stoyko's Tables opposite.

8 Lag in Audio Filter: Correction positive if filter causes a delay as is usual. 5

9 Personal Equation-Almucantars only: Where observer's Personal Equation is unknown, assume $PE = -0^s.088$. 9b

10 Diurnal Aberration-Almucantars only: $= -0^s.0213 \sin E$, where E = Elevation of the Almucantar. For Meridian Transit 4
Observations *see* Apparent Places, Table VII, and make no entry here. 9a

AZIMUTH

11 Eccentricity Correction: If applicable. 20

12 Convergence: If applicable = ($\Delta\lambda$ in secs of arc between Ecce and Trig) $\sin\varphi$. 20

13 Correction to BIH Pole: $\Delta A_p = (x \sin\lambda - y \cos\lambda) \sec\varphi$. 21

14 Diurnal Aberration: $+0.32'' \cos A \cos\varphi \sec E$. For Sigma Octantis $= -0.32''$. 22

15 Skew normals, or Elevation of RO: $= (108.3'' \, h_2 \sin 2A \cos^2\varphi) 10^{-6}$, where h_2 is height in metres above sea-level of the RO. 23

HEIGHT

16 All heights in metres on Australian Height Datum.

Notes:

LAPLACE EQUATION AND DEVIATION OF THE VERTICAL

The geodetic values in this section should be Australian Geodetic Datum co-ordinates. Corrections applicable to this section are dealt with comprehensively in paragraphs 25–29 inclusive of "Small Corrections to Astronomic Observations".

THE BIH POLE

The 1968 BIH Reference System Pole, Po, was chosen in 1970 (*see* Annual Report for 1970 of Bureau International de l'Heure) such that its mean deviation from the Conventional International Origin, CIO, for the period 1964.0 to 1967.0 was zero. It is maintained by examining the systematic and random error levels of participating observatories relative to the weighted mean solution.

(UT_1–UTC) AND DUT_1

Projected DUT_1, to one decimal place only, has been transmitted by VNG since 1 January 1972.

For all observations carried out after 31 December 1971, DUT_1 is to be incorporated in the chronometer error shown in the fieldbooks when recording latitude and azimuth observations.

Using this combined (chronometer–DUT_1) error in the calculations will ensure that the Preliminary Latitude and Preliminary Azimuth values referred to on the front of this summary are in terms of UT_1.

The Preliminary Longitude should be calculated in terms of UTC (ignore DUT_1).

Correction *5* of this summary ($\Delta t = UT_1 - UTC$) will relate the astronomic longitude to UT_1.

Apparent Velocity Propagation of Short Wave Radio Time Signals

ANNA STOYKO, BULLETIN HORAIRE, No. 10, 1956

Distance in km	Velocity in km/sec	Delay in milliseconds
1000	254 300	3.9
1500	258 300	5.8
2000	261 500	7.6
2500	264 200	9.5
3000	266 400	11.3
3500	268 200	13.1
4000	269 800	14.8
4500	271 200	16.6
5000	272 400	18.4
6000	274 300	21.9
7000	275 900	25.4
8000	277 200	28.9
9000	278 300	32.3
10000	279 200	35.8
11000	280 000	39.3
12000	280 600	42.8
13000	281 200	46.2
14000	281 800	49.7
15000	282 200	53.2
16000	282 600	56.6
17000	283 000	60.1
18000	283 300	63.5
19000	283 600	67.0
20000	283 900	70.4
21000	284 200	73.9
22000	284 400	77.4
23000	284 600	80.8
24000	284 800	84.3
25000	285 000	87.7
26000	285 200	91.2
27000	285 300	94.6
28000	285 500	98.1
29000	285 600	101.5
30000	285 800	105.0
31000	285 900	108.4
32000	286 000	111.9
33000	286 100	115.3
34000	286 200	118.8
35000	286 300	122.2
36000	286 400	125.7
37000	286 500	129.1
38000	286 600	132.6
39000	286 700	136.0
40000	286 800	139.5

PREPARATION OF HORIZONTAL CONTROL STATION SUMMARIES

1. GENERAL

1.1 Introduction

A uniform method, detailed below, is to be adopted for the preparation of station summaries for horizontal control stations. Firstly, a handwritten draft summary must be prepared. This should show all details except computer output and air photo identification data. The access sketch and RM diagram can be quickly but neatly drawn. The main object of the draft is to get all the data extracted accurately from the various field books and laid out in its correct position on the station summary form. This is to be methodically done as laid out in steps 2.1 to 2.12 of these instructions. Once the draft summary has been completed, and independently checked, the final summary can be drafted and typed.

1.2 Responsibility

1.2.1 The responsibility for the preparation of a final horizontal control station summary lies with the State organisation which is responsible for the geodetic survey of that State.

1.2.2 The responsibility for the preparation of a draft horizontal control station summary lies with the organisation by whom the station was established.

1.2.3 All members of the National Mapping Council will attempt to acquire information for stations established by private firms.

1.3 Station Summary Form

A station summary should be prepared for every permanently marked horizontal control station of potential use for topographic mapping or from which further geodetic surveys can conveniently be controlled. The summary form to be used is that approved by the National Mapping Council of Australia at Annex A.

2. GENERAL PROCEDURE FOR PREPARATION OF STATION SUMMARY

2.1 Station Number and/or Name

This is to be in upper case: eg JOHNSTON

2.2 Order of Station

This is to be in upper case: eg FIRST

2.3 Authority

The authority preparing the original station summary is to be shown in upper case: eg ROYAL AUSTRALIAN SURVEY CORPS

2.4 Original Station Established by:

The Authority which established the original is to be shown in upper and lower case: eg NSW Department of Lands 1881. Frequently the summary will have been prepared and the station established by two different authorities.

2.5 Existing Station Marked by:

The authority which marked the original station is to be shown in upper and lower case: eg Department of Crown Lands and Survey, Victoria 1972. If the station mark is adopted without replacement and a station summary has not been prepared previously, enter in this space the name of the authority which originally marked the station; details of the type of mark should be noted under 'Station mark'. If the position of the existing mark is adopted although the mark itself has been replaced by another authority, this latter authority should be shown as marking the existing station and that should be noted under 'Station mark'.

2.6 Reference Books

Number of field books used and authority abbreviations are to be shown.

2.7 State

The State in which the horizontal control station is established is to be shown in the correct space in upper and lower case.

2.8 Map Name, Map Number and Scale

Details of the relevant 1:250 000 map area to be entered in upper case in the appropriate spaces on the right hand side of the summary.

2.9 Access and Locality Sketch: Particulars of Station Marking and Beacon

In this section of the summary, descriptive notes are to be entered in the following order:

2.9.1 Station Mark

Wording should be along the following lines:

15 mm diameter copper tube, 150 mm long, set in concrete block 300 mm by 300 mm and 500 mm deep, stamped NTS204.

When a previously established station mark has been adopted this should be stated and the mark described. When a previously established station mark was located but not adopted, the connection to the old mark be shown. The height shown on the summary must refer to the station mark described.

2.9.2 Beacon

A simple description should be given, such as: "The beacon consists of a steel pole with vanes, erected in the centre of a rock cairn about 2.2 metres in diameter and 2 metres high. Steel pole is 65 mm square, 4.0 metres high. The four vanes are half discs 1.0 metres in diameter; the tops of the vanes are 4.000 metres above the station mark".

2.9.3 Reference Marks

Descriptive notes should be along the following lines:

"All RMs are 15 mm diameter copper tubes set in concrete blocks at ground level" or "RM1 is a 15 mm brass rod set in concrete block, RM2 is a Broad Arrow chiselled in solid rock, RM3 is the centre of a GI nail hit into a blaze on a Desert Oak". Special care is to be taken to fully describe and number reference marks which are more complicated than usual, ie, recovery marks where the station mark is on unreliable ground, or offshore stations, which can have various types of RMs, witness posts etc.

2.9.4 Access

The most important thing is whether the station is a drive-on or a walk-on; this should be clearly stated.

Other important aspects are those which cannot be shown on a sketch, eg: "There is a two-hour hard climb from the vehicle"; "The station is on the highest point of a prominent red bluff"; "The station is about 100 metres west of the highest point of a slight domed ridge"; "Vehicle access can be made right to the station mark - however great care must be taken on the steep slope over the last 400 metres".

2.9.5 Access Sketch

Often an extract from a published map can be annotated and stuck on the master copy. If a sketch is made, little in the way of descriptive notes of the access route should be necessary. It must always be drawn with north to the top of the page and with a simple north point to confirm this. The sketch need not be drawn to scale, the main consideration being to indicate clearly any particularly difficult sections of the route. Running distances, commencing with zero kilometres at a readily definable starting point and the final distance at the station, are to be shown, the figures to be written so they can be read as the sketch is oriented to the direction of travel.

2.9.6 Reference Mark Information

The relationship between reference marks and the station mark may be shown in the form of a table or a diagram. This information is to be shown at the bottom right hand side of the station summary. It should show true bearings to the nearest minute and horizontal distances to the nearest 0.005 metres, from station mark to each RM. It should also show distances between RMs. In the case of distant recovery marks true bearings to such marks should be shown to the nearest second of arc. If it is necessary to show any distances which have been computed but not measured, these must be marked "computed". Computed connections between RMs are useful and may be shown. RMs should be numbered as they are in the original field book; where no numbers were given, they should be numbered, in sequence, clockwise from north. Heights of RMs above (+) or below (-) the level of the station should be shown. No RM should be termed "the eccentric station" on a station summary. If re-monumentation has taken place at a station, and the new and old marks are not concentric, the relationship of the old mark to the new mark can be shown in a similar manner. The coordinates of the old mark may need to be shown also. See also paragraph 2.9.1.

If a diagram is preferred, it is to be oriented with north to the top of the summary.

2.10 Computer Output

To reduce the possibility of error, the station number and name, section and serial number should appear on the output. The work ECCE must not appear. The appropriate RM number should be used in the computer output to identify any mark from which eccentric observations were made, eg NM/B/133 RM2. Only the computer output of the current section in which the station was initially adjusted or computed should be affixed to the right hand side of the station summary in the place provided.

2.11 Levelling Connection

Details should be shown near the RM diagram, along these lines:

"Third order level connection, 1970". The height of point should be shown to three decimal places in the space provided on the computer output.

2.12 Air Photo Identification

Before entering any details of either spot or mapping photographs on the station summary, the available data should be examined with reference to the following:

2.12.1 Ensure that the point to which spot photography refers is clearly indicated on the diagram or in the description which appears on the station summary.

2.12.2 Coordinates of the photo reference point must be available and entered on the summary with identical numbering to that shown on the diagram.

2.12.3 The spot photography information is prefixed by the point number to which it refers, and should be entered in lower case on the line "Photo Identification", worded:

PRP NM/F/2006 - Spot photography 1968
held by Department of Lands and Surveys, Perth

or

PRP RM3 - positive mapping photography 1972
held by Division of National Mapping

or

Station mark - Spot photography 1978
held by CMA

2.12.4 The authority holding the original photos and negatives should ensure that:

i) The spot photographs are clearly labelled with the exact reference number of the photo reference point, station mark, film number, exposure number of the negatives, year of photography and scale of photography.

ii) Where an identification is made on, or subsequently transferred to, mapping photography, the ident photograph shall be clearly labelled with the precision of the identification, eg:

Precise ident - transferred from spot photography

or

Precise field ident - see description on the back of the photograph.

or

Area ident only - not to be used for photogrammetric purposes.

3. DOPPLER SATELLITE FIXES

3.1 If Doppler satellite observations have been made at a conventional horizontal control survey station, a note at the bottom left hand side of the existing station summary is to be entered: eg DOPPLER 1 1978; a copy of the Doppler station analysis sheet may be attached or filed separately.

3.2 A summary should also be prepared for any other permanently marked station at which Doppler satellite observations have been carried out - see Annex E.

3.3 The general procedure for preparation of station summaries should be followed for such stations.

3.4 The order of accuracy is to be in upper case, eg DOPPLER 1, 2, 3, 4, 5 or 6 as determined by the classification given in Specification 3.4.

3.5 Doppler-derived AGD/AMG coordinates should be shown to .01"/1 metre together with the uncertainty of coordinates and the month of year of the Doppler satellite observations. If a levelled height exists, this is to be shown and so labelled.

4. DISTRIBUTION

A clear copy of each new station summary should be supplied to the Chairman of the National Mapping Council for distribution to other National Mapping Council members and the public on request.

ANNEX D

UPDATING OF HORIZONTAL CONTROL STATION SUMMARIES

1. GENERAL

1.1 Introduction

Amendment and alteration to a station summary may be required following a visit by any survey organisation in Australia.

1.2 Responsibility

The State organisation responsible for geodetic surveys is the controlling authority for all permanently marked geodetic stations within the State. It is responsible for updating summaries, but must be supplied with information from the organisation whose surveyors visited the station.

2. GENERAL PROCEDURE FOR UPDATING STATION SUMMARIES

2.1 To keep summaries for permanently marked horizontal control stations up to date, the following procedures are to be implemented:

2.1.1 A non-State authority which revisits a station should notify the Division of National Mapping as soon as practicable of any changes to the addition or correction to ground mark, reference marks, beacon description, descriptions of access and diagrams, details of any level connection, details of spot photography, date of visit. The date of the visit to the station is to be entered on the bottom left hand side of the station summary, as follows: "Station revisited, 1975 Australian Survey Office, Perth".

If reference marks are not numbered on a copy of the original station summary they should be numbered in sequence clockwise from north.

2.1.2 The amendments mentioned in paragraph 2.1.1 should be shown in red ink on a copy of the station summary, the same as that used in the field and supplemented by extracts from field books.

2.1.3 Copies of the above amendments will be forwarded by the Division of National Mapping to the relevant State organisation.

2.1.4 In the case where amendments are not clear they should be resolved between the authority which revisited the station, the Division of National Mapping and the relevant State authority.

2.1.5 Computer output for subsequent adjustments, for which the coordinates of stations remain unchanged, will not be added to the summaries. Only the name or number of the adjustment and the serial number of the station should be added.

3. AIR PHOTO IDENTIFICATION

The procedure to be followed is as laid down in Annex C paragraph 2.12.

4. DOPPLER SATELLITE FIXES

The procedure to be followed is as laid down in Annex C paragraph 3.

5. DISTRIBUTION

A clear copy of each amended or revised station summary should be supplied to the Chairman of the National Mapping Council, for distribution to other National Mapping Council Members and the public on request.

NMP/80/190.3

NATIONAL MAPPING COUNCIL OF AUSTRALIA
STATION SUMMARY

Serial No

Authority **DIVISION OF NATIONAL MAPPING**

Station Number and Name: **NM / F / 711** Order: **DOPPLER 2**

Original Station Established by: **Division of National Mapping** Date: June, 1978

Existing Station Marked by: **Division of National Mapping** Date: June, 1978

Reference Books: **JMR: Station Log Collier 3 , Level: 7181 , Spot Photography: NM 16140**

Cadastral Location: State **Western Australia** County/District

Parish/Hundred Allotment/Section/Portion

Map Name: **COLLIER** Map Number: SG 50.4 Scale 1: 250,000

DATUM: Australian Geodetic Datum 1966

RECTANGULAR COORDINATES. Australian Map Grid. In Metres

GRID BEARING = ADJ AZIMUTH + CONVERGENCE

HEIGHTS: In Metres on the Australian Height Datum

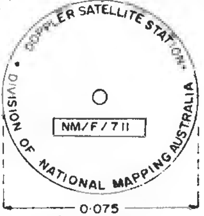
SOUTH LATITUDE	EAST LONGITUDE	ZONE	EASTING	NORTHING	CONVERGENCE	HEIGHT
24 46 46.145	119 35 52.246	50	762673.58	7256962.74		594.256
		51	155946.14	7255175.11		

Doppler Satellite observations June 1978. Uncertainty of derived AGD coords ± 3.5 metres

Access and Locality Sketch:

Particulars of station marking and beacon

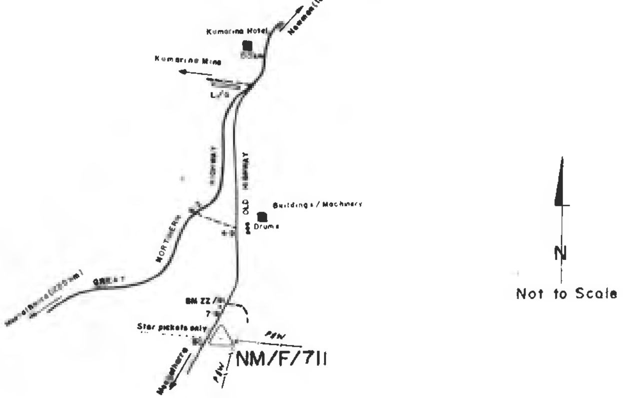
Station Mark : 0.075m diameter bronze plaque, inscribed as below, set in concrete block (painted white). Mark surrounded by 2m radius annulus of white flat stones. Mark is located between two white star picket fence posts.



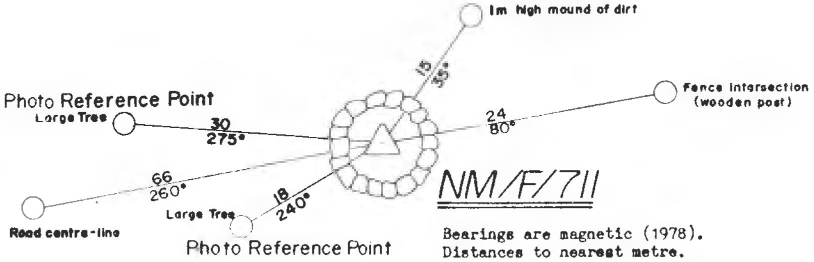
Beacon : Nil

Reference Marks : See reference mark diagram

Access : **From Kumarina Hotel (approximately 180km south of Newman) continue along Great Northern Highway(new) past bend to south-west until turn-off (easterly- 4.2km) reached. Continue along this for 0.7km to 'T' intersection(drums bar continuation). Turn right, proceed along old highway alignment for 3.1km until east-west fence-line reached (near BM ZZ/91). Mark is approximately 60m east of road along fence-line.**



Cadastral Connections & Reference Marks.



Bearings are magnetic (1978).
Distances to nearest metre.
Third Order level connection 1978.

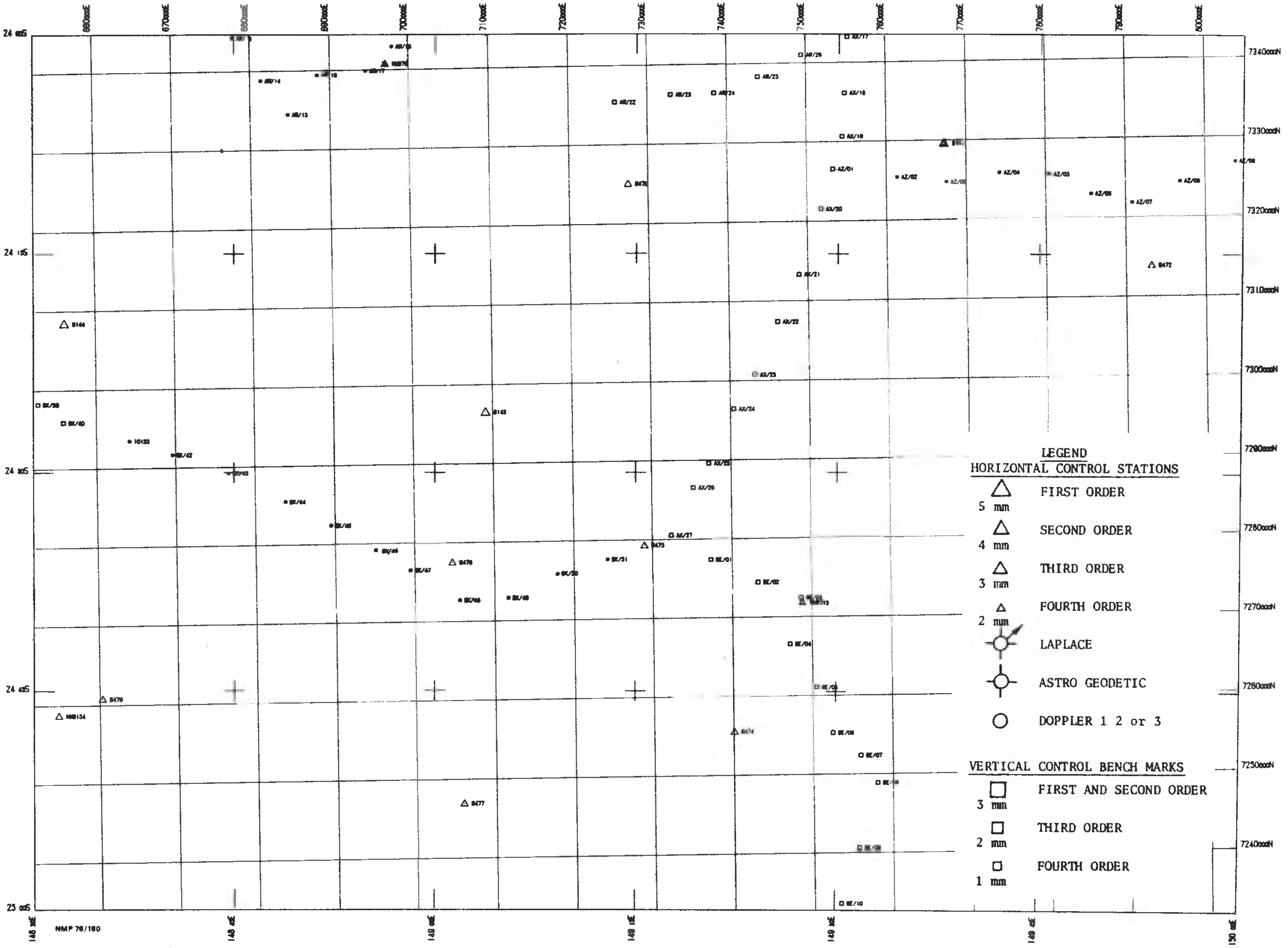
Photo Identification: 1978 **Spot Photography : NM / F / 711 CAF 8358 Exp 5-13.**
1972 Mapping Photography : Collier CAF 7545 Run 7/160.

Certified free of transcription errors: Date: 19/10/78

Approved by: DIRECTOR DIVISION OF NATIONAL MAPPING Date:

AHQ/DET/1094

Annex E

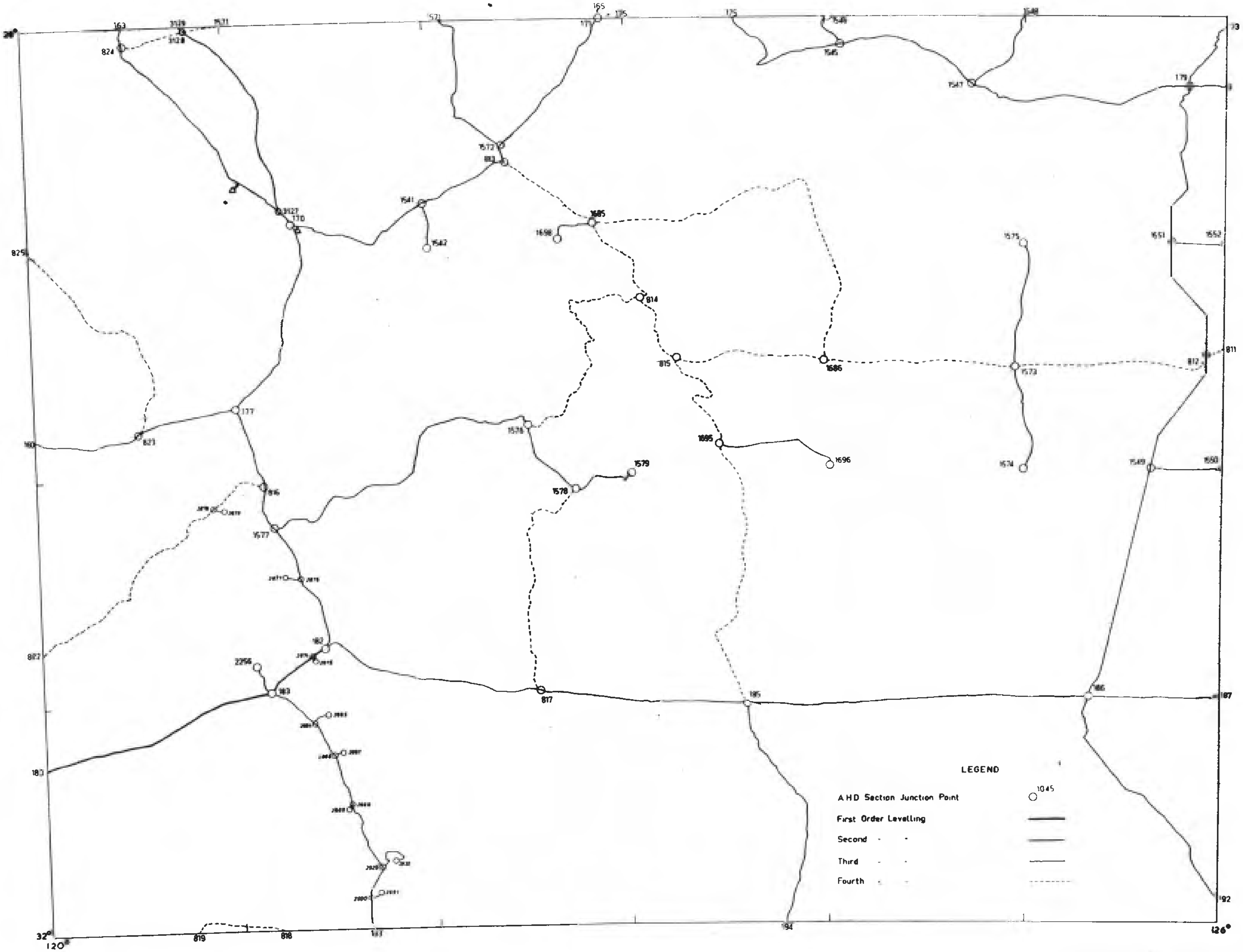


BARALABA SG 55-4

NMP/80/190-5

DIVISION OF NATIONAL MAPPING

AUSTRALIA
VERTICAL CONTROL



Annex G

SH 51

KALGOORLIE 3352

PRINTED 5/11/1981

INFORMATION SUMMARY

Produced by National Mapping from the
NATIONAL GEODETIC HORIZONTAL CONTROL DATA BASE
on behalf of the
National Mapping Council

MAURICE
B 017

[961 LAST UPDATED 9/81

TYPE : TRIG/TRAV STN.

GEOGRAPHICAL COORDINATES : AUSTRALIAN GEODETIC DATUM
HEIGHT : AUSTRALIAN HEIGHT DATUM
UNLESS OTHERWISE STATED
RECTANGULAR COORDINATES : AUSTRALIAN MAP GRID
HEIGHT & AMG ARE IN METRES

SOUTH LATITUDE	EAST LONGITUDE	HEIGHT
23 56 09.9257	151 14 29.1007	224.442

ZONE	EASTING	NORTHING
56	321024.769	7351725.356

MAPS : 1:250000 F56-13 1:100000 9150

THIS POINT IS IN THE STATE OF QLD
THIS POINT IS OF INTEREST TO QLD

	ACCURACY	DATUM	
HORIZONTAL	FIRST	AGD	
VERTICAL	LEVELLED	AHD	UNCONFIRMED

SUBSEQUENT OBSERVATIONS : AERODIST. DOPPLER. LAPLACE.

RELATED INFORMATION

	PRIMARY STATION
PREVIOUSLY	IN 82-32
ECCE TO	MAURICE MWT

OLD VALUES : CLARKE.

YEAR LAST VISITED : 1975
STATION MARK : BRONZE PLAQUE IN CONCRETE AT SURFACE
BEACON/CAIRN : THE ADJACENT STRUCTURE MAY BE USED AS AN ECCENTRIC BEACON
ACCESS : 2 WD VEHICLE, 0 HOURS 0 MINUTES CLIMB ON TIME.
SUMMARY : AUTHOR: DNM YEAR:1976

HORIZONTAL ADJUSTMENT NAMES/SERIALS

ORIGIN OF COORDINATES : 32-34/24

MONTON/7.82-32A/51.ADSBA23/71.CURTIS/108.ADSBA6/21.BRISROCK/60.32-34/24

VERTICAL ADJUSTMENT NAMES

QLD LANDS

PHOTO IDENTIFICATION
RA SVY

NMP/80/190.7

PRINTED [illegible]

1:100 000 MAP LIST

Produced by National Mapping from the
NATIONAL GEODETIC HORIZONTAL CONTROL DATA BASE
on behalf of the
National Mapping Council

COMPLETE LIST for sheet 8259

(* STATION DESTROYED)

STATION NAME	ALTERNATIVE REFERENCES		HORIZ ORD	DAT	LATITUDE	LONGITUDE	ZN	EASTING	NORTHING	HEIGHT	VERTICAL ORD	DAT
BIG JACK	C 369		3RD	AGD	19 24 40.0020	146 48 09.4879	55	479278.850	7853663.663	350.08	TRG	AHD
BLACK	C 364		3RD	AGD	19 16 54.4781	146 33 41.1547	55	453918.489	7867925.983	410.48	TRG	AHD
BOHLE	B 046	NM/B/316	* 1ST	AGD	19 16 03.3000	146 41 31.0127	55	467629.495	7869528.544	128.30	TRG	AHD
CASTLE HILL	B 527		* 2ND	AGD	19 15 35.9329	146 48 05.2747	55	479136.791	7870386.504	285.70	TRG	AHD
CASTLE HILL	C 363		3RD	AGD	19 15 34.9720	146 48 11.7194	55	479324.882	7870416.253	281.482	LEV	AHD UN
CATARACT	B 530		2ND	AGD	19 17 15.3752	146 30 32.0196	55	448399.950	7867268.870	721.30	TRG	AHD
JACK	C 368		3RD	AGD	19 23 59.3186	146 50 48.3234	55	483909.977	7854918.861	228.03	TRG	AHD
KULBURN	C 362		3RD	AGD	19 14 03.3037	146 38 01.3301	55	461501.193	7873204.977	139.14	TRG	AHD
LOUISA			4TH	AGD	19 16 49.7352	146 44 29.1089	55	472830.146	7868109.758	193.54	TRG	AHD
MARLOW	B 531		2ND	AGD	19 11 36.6145	146 44 23.4664	55	472651.060	7877733.792	213.10	TRG	AHD
MUNTALUNGA	C 365		3RD	AGD	19 20 49.4911	146 52 48.3086	55	487405.381	7860756.342	228.36	TRG	AHD
PINNACLE	B 524		2ND	AGD	19 24 27.3159	146 38 10.1082	55	461797.765	7854025.149	729.20	TRG	AHD
RATTLESNAKE	B 532		2ND	AGD	19 02 03.8075	146 36 42.3204	55	459145.332	7895314.947	123.40	TRG	AHD
ROUND	B 519		2ND	AGD	19 27 41.0146	146 41 40.6498	55	467948.697	7848083.270	374.90	TRG	AHD
SAUNDERS	B 499		2ND	AGD	19 11 55.1957	146 35 47.3185	55	457579.513	7877133.964	90.30	TRG	AHD
SISTER	B 515		2ND	AGD	19 25 03.5730	146 54 17.5452	55	490013.168	7852948.266	428.00	TRG	AHD
STUART	B 525		2ND	AGD	19 20 41.5029	146 46 46.2409	55	476841.692	7860991.479	504.60	TRG	AHD
TABLELAND	B 523		2ND	AGD	19 26 35.9349	146 31 10.5518	55	449572.473	7850041.717	642.00	TRG	AHD
TOWNSVILLE	DEN 1		2ND	AGD	19 15 30.4768	146 42 56.0532	55	470110.128	7870541.658	6.744	LEV	AHD
WOODSTOCK HILL	C 366		3RD	AGD	19 21 42.3196	146 59 03.3711	55	498347.998	7859136.855	234.77	TRG	AHD

Annex I

```
        60  STATIONS LISTED
                                                                                          PRINTED  5/11/1981
                                        ADJUSTMENT LISTING

                                  Produced by National Mapping from the
                               NATIONAL GEODETIC HORIZONTAL CONTROL DATA BASE
                                         on behalf of the
                                     National Mapping Council

                                                                                      section 17-15
THERE ARE    8 STATIONS IN THIS SECTION
NAME(S)                                         SECTION(S)     ORDER      LATITUDE          LONGTITUDE         HEIGHT


NM B  38                                                         1       25 36 31.4978     141 59 21.2100      192.405
                                                NM740D1/3
                               ORIGINAL COORDINATES  17-15/6

NM B  60                                                         1       26 14  7.8686     142 39 10.6709      297.000
                               ORIGINAL COORDINATES  17-15/2

NM B  36                                                         1       26  5 31.5409     142 34 22.4936      234.600
                               ORIGINAL COORDINATES  17-15/4

NM B  37                                                         1       25 56 17.8357     142 23 56.3507      164.400
                               ORIGINAL COORDINATES  17-15/5
                                                ADSBA16/67

HOWITT                     NM B 34                               1       26 32  9.3642     142 23   .5596      202.575
                               ORIGINAL COORDINATES  16-17/35
                                                17-15/1
                                                17-93/1
                                                ADSBA15/2
                                                ADSBA16/77

NM B  35                                                         1       26 10 32.4182     142 39 31.7876      279.300
                               ORIGINAL COORDINATES  16-17/36
                                                17-15/3
                                                17-93/5

BUTLER                     NM B  39                              1       25 25 32.7466     141 47 27.0962      298.200
                               ORIGINAL COORDINATES  10-15/13
                                                15-18/1
                                                17-15/10
                                                NM74QD1/1

NM B  40                                                         1       25 16 13.3996     141 50 18.6072      299.700
                               ORIGINAL COORDINATES  10-15/14
                                                15-18/2
                                                17-15/7
```